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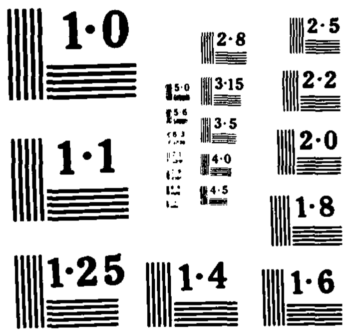

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TECHNICAL REPORT HL-83-13

NORFOLK HARBOR AND CHANNELS DEEPENING STUDY

Report 2

SEDIMENTATION INVESTIGATION

Cheasapeake Bay Hydraulic Model Investigation

by

R. C. Berger, Jr., Samuel B. Heltzel, Robert F. Athow, Jr.
David R. Richards, Michael J. Trawle

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631
Vicksburg, Mississippi 39180-0631

March 1985

Report 2 of a Series

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

[illegible] report presents the sedimentation findings from combined physical [illegible] (hybrid modeling) of deepening the approach channels [illegible] Newport News, Virginia. Because of the varying nature of [illegible] material from the project navigation channel, the tests included two [illegible] sediment transport models, which were referred to as the [illegible] model and the Elizabeth River model. The sediment along the

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20. ABSTRACT (Continued).

Thimble Shoal portion of the navigation channel consists predominantly of non-cohesive material while the sediment along the Elizabeth River portion of the navigation project consists primarily of clays and silts. Sedimentation in a third portion of the overall project, referred to as the Atlantic Ocean Channel, was evaluated analytically without using a numerical sediment transport model.

Based on sedimentation results from the Elizabeth River numerical model, the increase in shoaling caused by channel deepening as proposed will be 23 percent. The distribution of shoaled material will not be significantly altered, other than a slight increase in skewness toward the downstream end.

Based on sedimentation results from the Thimble Shoal numerical model, the increase in shoaling caused by channel deepening as proposed will be about 20 percent. The distribution of shoaled material will be slightly altered in that both the upper and lower channel shoaling peaks which presently exist will tend to migrate even more toward the ends of the dredged channel.

Based on the analytic analysis, the estimate of shoaling for the new Atlantic Ocean Channel is about 200,000 cu yd annually.

PREFACE

In December 1981, the US Army Engineer Waterways Experiment Station (WES) was requested by the US Army Engineer District, Norfolk, to conduct an investigation of possible sedimentation changes in Norfolk Harbor and Channels caused by the proposed deepening.

The study was conducted by personnel of the Hydraulics Laboratory, WES, under the general direction of Messrs. H. B. Simmons, retired former Chief of the Hydraulics Laboratory, F. A. Herrmann, Jr., Chief of the Hydraulics Laboratory, R. A. Sager, Chief of the Estuaries Division, E. C. McNair, Chief of the Sedimentation Branch, and R. A. Boland, Chief of the Hydrodynamics Branch. The project was conducted and this report prepared by Messrs. R. C. Berger, Jr., S. B. Heltzel, R. F. Athow, Jr., D. R. Richards, and M. J. Trawle. Other WES personnel participating in the study were Messrs. J. A. Boyd, D. M. Marzette, and Ms. V. P. Pankow.

We gratefully acknowledge the valuable contributions of Dr. J. C. Ludwick of Old Dominion University and Mr. J. R. Melchor, US Army Engineer District, Norfolk.

Commanders and Directors of WES during the conduct of this study and preparation and publication of this report were COL Tilford C. Creel, CE, and COL Robert C. Lee, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, US CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

US customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acres	0.4047	hectares
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	25.4	millimetres
knots (international)	0.514444	metres per second
miles (US nautical)	1.852	kilometres
miles (US statute)	1.609344	kilometres

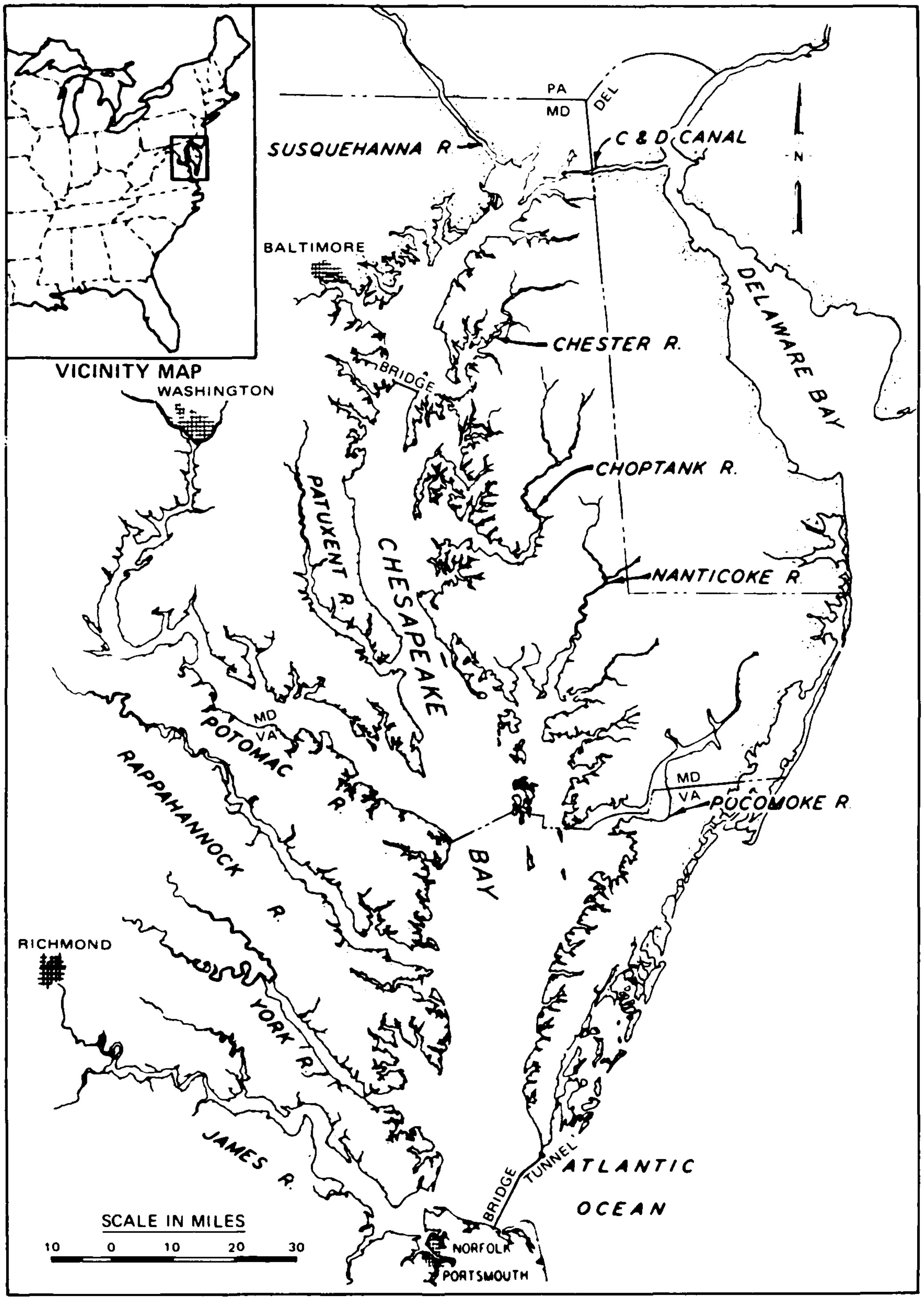

Figure 1. Location map

NORFOLK HARBOR AND CHANNELS DEEPENING STUDY

SEDIMENTATION INVESTIGATION

Chesapeake Bay Hydraulic Model Investigation

PART I: INTRODUCTION

Chesapeake Bay

1. Chesapeake Bay with its tributary estuaries forms the largest estuarine system in North America. The 190-mile*-long estuary varies in width from 4 to 30 miles with an average depth of 28 ft (Figure 1). The mean annual discharge of its 126 freshwater tributaries is approximately 70,000 cfs, almost 90 percent of which is contributed by the Susquehanna, Potomac, Rappahannock, York, and James River basins. The Atlantic Ocean provides salt water to the bay, producing large salinity variations within its boundaries. The eastern shore is generally saltier than the western shore, attributed in part to the dominance of freshwater flow from the western shore tributaries and to the counterclockwise tendency of flow resulting from Coriolis force (Richards and Morton 1983).

2. Chesapeake Bay is classified geologically as a drowned river valley estuary. The Holocene sea-level rise inundated the Susquehanna River Valley to form the bay. Sedimentation from the tributaries as well as erosion of the banks has contributed to maintaining the bay's broad, shallow character. The bay is classified as a partially mixed estuary, although various stages of freshwater discharge and tidal and wind mixing cause portions to alternate between well mixed and highly stratified. Tides are semidiurnal with mean ranges from 1 to 2 ft. The length of Chesapeake Bay is such that a complete tidal wave is contained within its limits at all times. Wind-generated waves are generally less than 3 ft in height, but larger waves can occur during high wind conditions. Average maximum velocities for tide and wind-driven currents range from 0.5 to 3 fps (Richards and Morton 1983).

3. Chesapeake Bay and adjoining tributaries are gradually filling with

* A table of factors for converting US customary units of measurement to metric (SI) units is presented on page 3.

sediment. The sources and losses of suspended material within this estuarine system are complex and varied. The sources of this sediment include the Susquehanna River, shore erosion, the ocean, and biological production. In the upper reaches of the bay, upland discharge is the principal source of sediment. However, shore erosion is the major source of sediment in the middle and lower reaches of the estuary. These processes may operate according to a seasonal cycle, so that at any time during the year, one of these processes may dominate the others. For instance, the Susquehanna River discharges 70 percent of its total annual sediment load during the spring freshet. In some cases, during heavy spring floods, the river will discharge the bulk of its sediment load in a few weeks. Analysis of sediment near the bay mouth indicates mainly sandy sediment. Sediments from tributary rivers (i.e., James River) are predominant silt and clay with progressively coarser sediment toward the bay. The tributaries contribute little if any sand. Most of the coarse sediment is either eroded from the margins of the lower and middle portions of the bay or possibly, according to some opinions, transferred from the ocean to the bay. Overall bay deposition averages 0.8 mm per year.

Norfolk Harbor

4. Navigational uses of Chesapeake Bay in the Norfolk area are of great importance to the Nation and the local communities. Due to its naturally protected harbors, the Norfolk area has historically been the home port of naval activities since colonial times. Commercially, Norfolk has played a major role in east coast bulk shipping for many years. Its closeness to the Appalachian coal fields and connecting rail lines has helped it become the largest coal exporting port in the United States. However, with the current trends toward deeper draft bulk cargo vessels and an ever-increasing demand for United States coal, Norfolk may lose some of this competitive advantage. Currently, several vessels calling on Norfolk must carry partial loads to navigate through the existing channels. Since the majority of the cargo passing through Norfolk is high in volume and low in price, the efficient use of shipping is crucial to bring profits. Unless the harbor is deepened, future deep-draft vessels may be forced to use other ports (Richards and Morton 1983).

Proposed Channel Improvements

5. The proposed improvements to channels and anchorages approaching

Norfolk Harbor are shown in Figure 2 and described as follows (USAED, Norfolk, 1980):

a. Increasing the depth of Thimble Shoal Channel from 45 to 55 ft below mean low water over its existing 1,000-ft width.

b. Increasing the depth of Norfolk Harbor Channel from 45 to 55 ft below mean low water over its existing 800- to 1,500-ft width to the coal terminal at Lamberts Point.

c. Increasing the depth of the Channel to Newport News from 45 to 55 ft below mean low water over its existing 800-ft width to the coal terminal at Newport News.

d. Dredging a new channel, referred to as the Atlantic Ocean Channel, off Virginia Beach to a depth of 57 ft below mean low water and a width of 1,000 ft over a length of 10.6 miles.

e. Constructing three fixed-mooring anchorage facilities, each capable of accommodating two large vessels simultaneously.

f. Increasing the depth of the Elizabeth River and the Southern Branch of the Elizabeth River between Lamberts Point (river mile 9) and the Norfolk and Western Railway Bridge (river mile 15) from 40 to 45 ft below mean low water over its existing 375- to 750-ft width.

g. Increasing the depth of the Southern Branch of the Elizabeth River between the Norfolk and Western Railway Bridge (river mile 15) and the US Routes 460 and 13 highways crossing (river mile 17.5) from 35 to 40 ft below mean low water over its existing 250- to 500-ft width, and providing a new 800-ft turning basin at the terminus of the channel improvement.

6. The depths listed in paragraph 5 are project depths and do not provide for advance maintenance or dredging tolerance. The actual depths for the proposed new channels with the advance maintenance and dredging tolerance should be 3 ft deeper. Previous deepening projects in the vicinity also had provisions for similar amounts of advance maintenance and dredging tolerance. The numerical model study of the proposed channel deepening used the existing project channel depths plus an additional 3 ft to allow for advance maintenance and dredging tolerance as the base condition; the plan condition used the deepened project depths plus 3 ft for advance maintenance and dredging tolerance (Richards and Morton 1983).

Purpose

7. The purpose of this study was to investigate the impact of the proposed channel deepening on the sedimentation characteristics within the

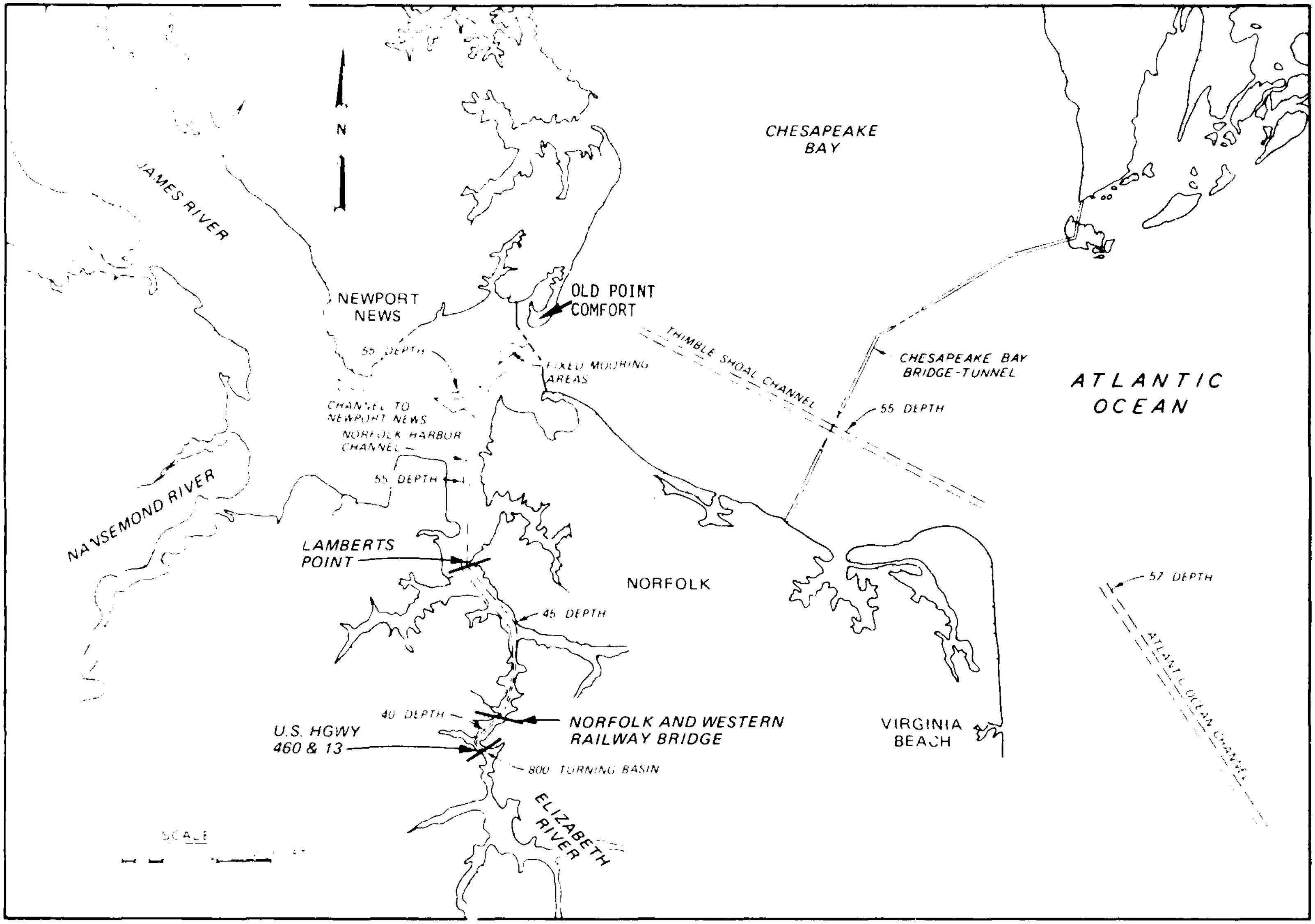

Figure 2. Project map showing proposed Norfolk dredged channels

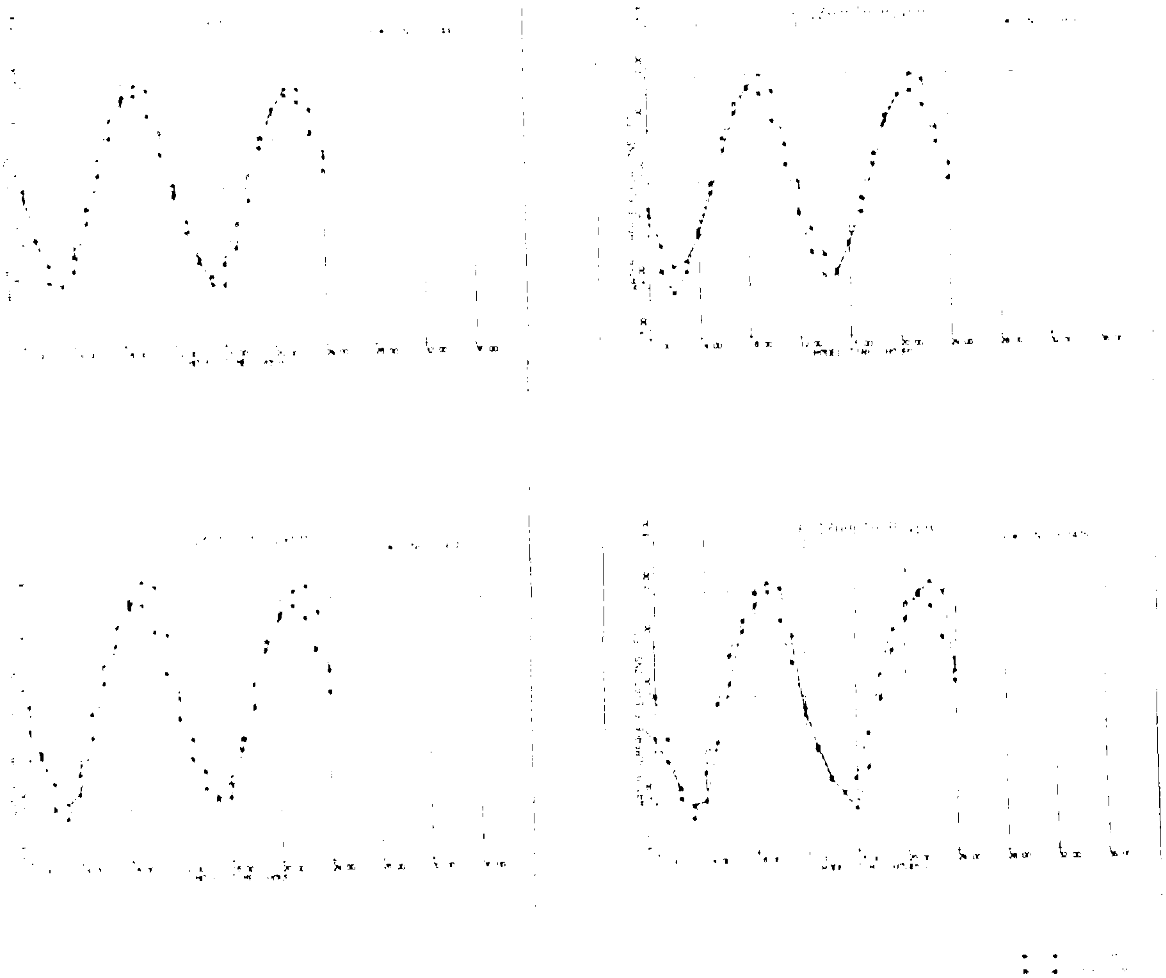

Figure 7. Elizabeth River water-surface elevations comparison (Base 1), physical model and numerical model

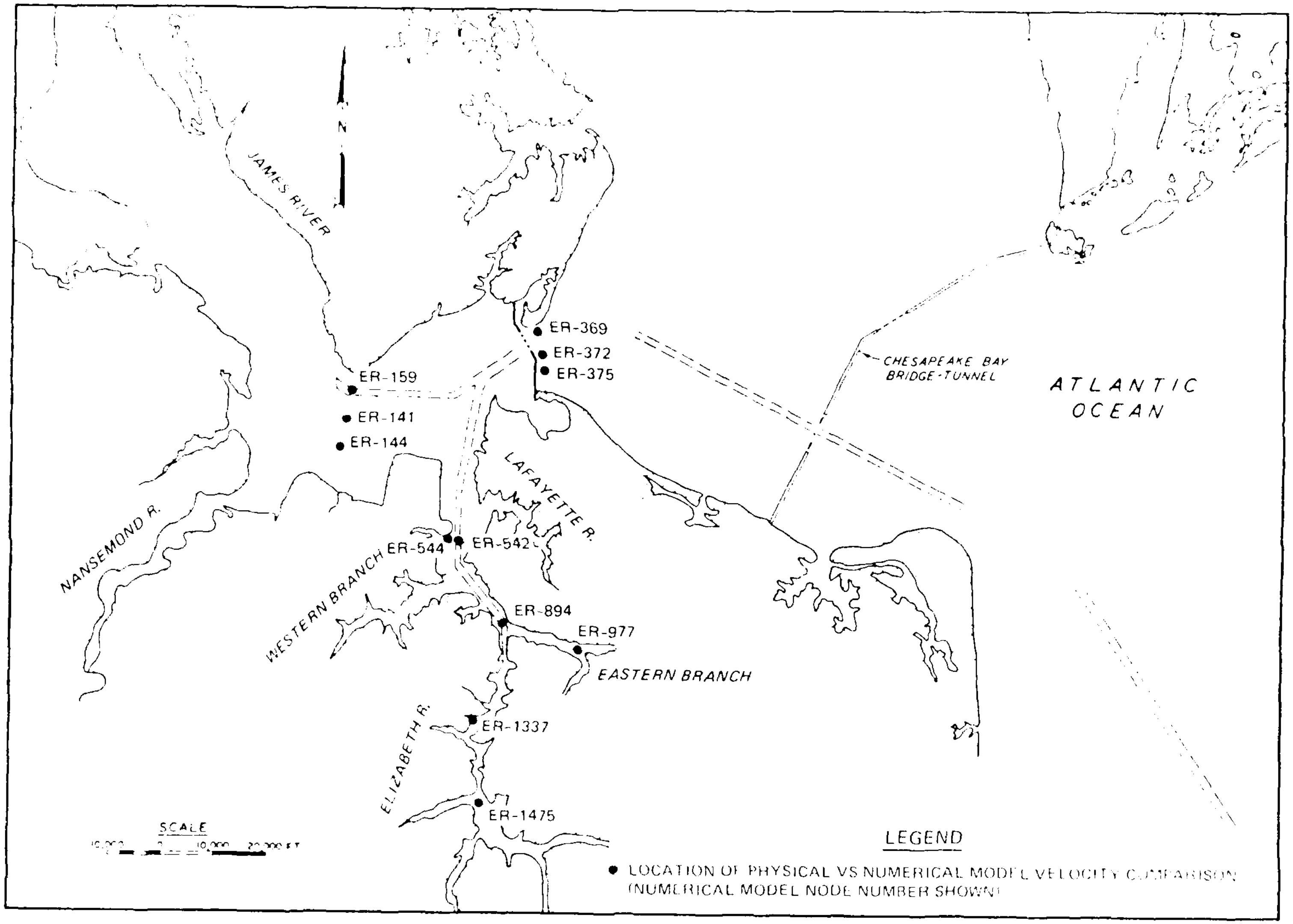

Figure 6. Location and node number of numerical model nodes used for comparison with physical model velocity stations, Elizabeth River (ER)

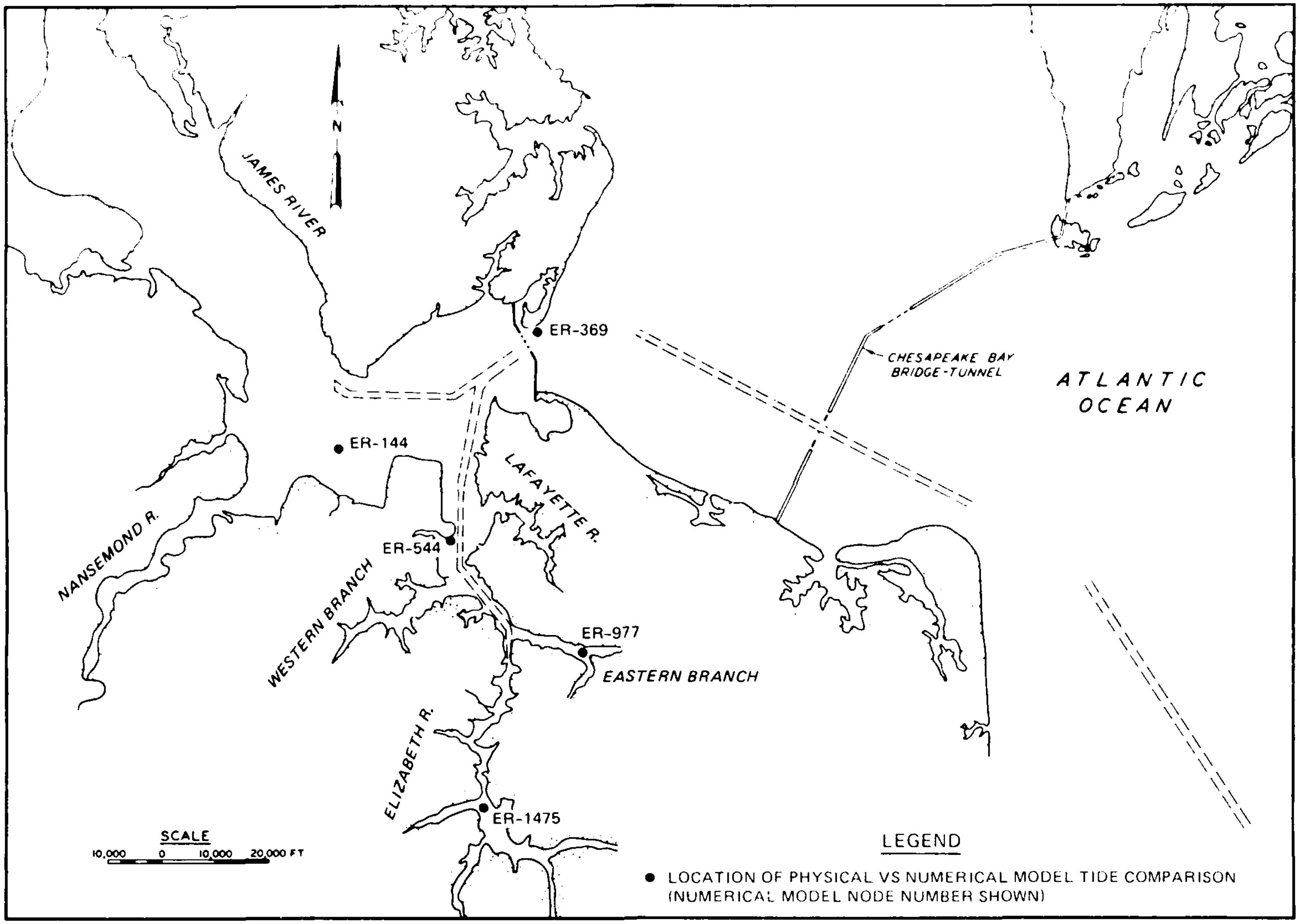

Figure 5. Location and node number of numerical model nodes used for comparison with physical model tide stations, Elizabeth River (ER)

mesh were modeled in the following manner. The upstream James River limit was a velocity or discharge boundary, as was the Elizabeth River Western Branch. The southern portion of Chesapeake Bay was an elevation or tidal boundary. The final water boundaries were the locks on the South Branches of the Elizabeth River, represented as a zero-velocity boundary.

31. Report 1 of this series discussed the results of the physical model testing program. Four test conditions were developed as representative of conditions commonly occurring in the prototype. Velocity and tidal elevations were recorded over a 25-hr period for each test condition for use in the numerical simulations. The test conditions are summarized below:

Condition	Discharge, cfs	Tidal Range, ft
1	200,000	4.8
2	200,000	3.0
3	70,000	4.8
4	70,000	3.0

The discharge shown above was the total freshwater discharge into the Chesapeake Bay from all tributaries. The tidal range was measured at the Atlantic Ocean physical model control gage. Figures 5 and 6 show the locations of the physical model velocity and water-surface elevation stations and their respective nodal mesh designations.

32. Initial calibration of the numerical model consisted of comparison of Base 4 (existing channel conditions with the boundary conditions of condition 4) numerical water-surface elevations and velocities with the data measured in the physical model at the same locations. If large differences were found between the numerical and physical model results, the numerical model parameters of either friction or eddy diffusivity were adjusted and the numerical model rerun. After satisfactory agreement had been achieved at all the critical stations for the Base 4 condition, the model was verified to the Base 1 condition. After the numerical model was considered verified, numerical simulations were conducted for Base 2 and 3 conditions to complete the series. Figures 7, 8, 9, and 10 are representative numerical model-physical model velocity and water-surface elevations comparisons at four critical locations. The numerical model water-surface elevations and velocities generated at each

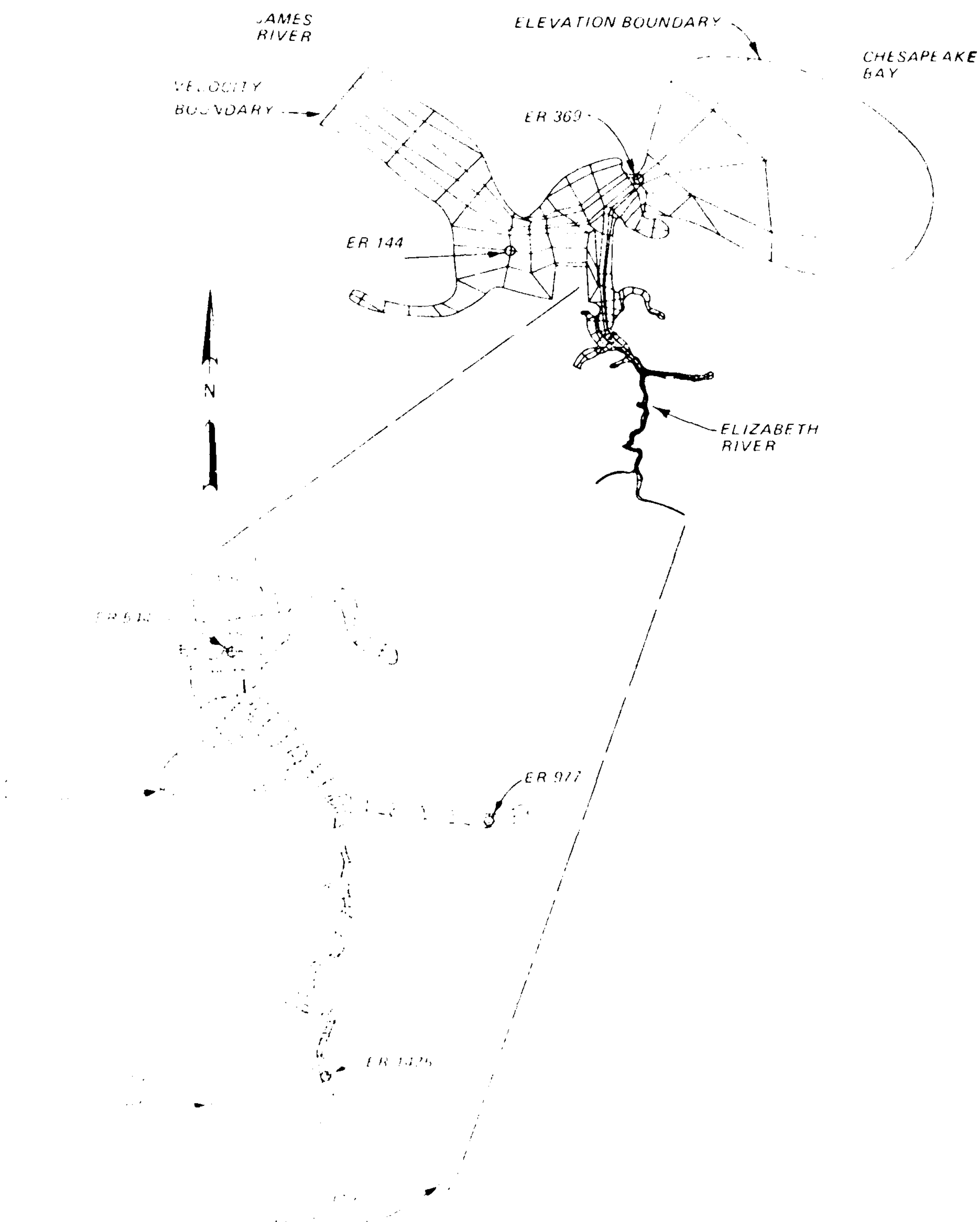

Figure 5. Elizabeth River mesh for numerical model

PART III: MODELING PROCEDURES

Elizabeth River

Hydrodynamic simulation

27. The first step in the numerical hydrodynamic simulation of the prototype was the development of a representative grid or mesh. Secondly, the physical model data were examined for completeness and consistency, and then the boundary condition files were generated for use in the time-stepping simulation. The final steps included the calibration of the numerical hydrodynamics by means of comparison and adjustment for best fit to the physical model stations in the interior portions of the numerical mesh, and the simulation of the deepened channel configuration for all boundary conditions.

28. Mesh development incorporated the physical boundaries of the estuary with a representation of the bathymetry. A mesh overlay was drawn on the most recent NOS charts available. The following table shows the charts used and their respective scales.

NOS Chart No.	Scale	Date
12253	1:20,000	Mar 79
12248	1:40,000	Nov 74
12222	1:40,000	Apr 81

The mesh intersections and midside connections between intersections were then numbered as nodes, and the mesh inclosures were numbered as elements. The Elizabeth River mesh was constructed of 1,496 computational nodes and 407 elements. Figure 4 shows the resulting mesh.

29. The physical model data were supplied to the authors in the form of magnetic tape and a tabular printout. The tape was read into the mainframe computer system used during the study (Boeing Computer Services) and the format was changed as required. A preprocessing code (CODE 24) was then used to average the point-depth values to yield depth-integrated values needed for the hydrodynamic code. The resulting files were labeled "boundary files" and were used as boundary updates for the time-stepping hydrodynamic simulations.

30. The upstream and downstream water boundaries of the Elizabeth River

$$S = \frac{P}{D}\left(\frac{\tau_b}{\tau_e} - 1\right) \tag{8}$$

where

P = erosion rate constant

τ_e = critical shear stress for particle erosion

26. STUDH is an outgrowth of the model SEDIMENT II (Ariathurai, MacArthur, and Krone 1977) developed under the direction of R. B. Krone at the University of California, Davis.

22. The value of Ceq can be determined from any of several transport relations. The sand version of the sediment model, STUDH, uses the [illegible]-White formula (1973), which performed satisfactorily in tests by White and others (White, Milli, and Crabbe 1975, Swart 1976). The transport potential is [illegible]lated to sediment and flow parameters by the following expressions [illegible] Ackers-White formulas.

23. Transport potential is converted to transport capacity by satisfying the availability constraint

$$GS_i = GP_i \cdot PI_i \tag{5}$$

where

- GS = transport capacity
- i = number of grain-size class
- GP = transport potential
- PI = 1/100 percent at bed surface covered by grain size, class i (the percent parameter varies from 100 to 0 and is based on weight of sediment as it is compatible with multiple grain-size class concepts)

24. Clay transport. Deposition rates for clay beds were calculated with the equations of Krone (1962).

$$S = \begin{cases} \dfrac{-2Vs}{D}\, C \left(1 - \dfrac{\tau_b}{\tau_d}\right) & \text{for } C < C_c \quad (6) \\[2ex] \dfrac{-2Vk}{D}\, C^{5/3} \left(1 - \dfrac{\tau_b}{\tau_d}\right) & \text{for } C > C_c \quad (7) \end{cases}$$

where

- Vs = fall velocity of a sand grain
- D = flow depth
- C = concentration of sediment
- τ_b = bed shear stress
- τ_d = critical shear stress for deposition
- C_c = critical concentration = 300 mg/ℓ
- $Vk = Vs/C_c^{4/3}$

25. Erosion rates were computed by a simplification of Partheniades (1962) for particle-by-particle erosion. The source term is computed by

w = flow velocity in z-direction, m/sec

z = direction perpendicular to x, m

D_x = effective diffusion coefficient in x-direction, m^2/sec

D_z = effective diffusion coefficient in z-direction

α_1 = a coefficient for the source term, 1/sec

α_2 = the equilibrium concentration portion of the source term, $kg/m^3/sec$

19. Bed shear stress. The bed shear stress, τ_b, takes the form:

$$\tau_b = \rho u_*^2 \tag{2}$$

where

ρ = water density

u_* = shear velocity

20. The Manning shear stress equation. The Manning form of the shear stress equation was used in this study

$$u_* = \frac{un}{CME} \frac{\sqrt{g}}{D^{1/6}} \tag{3}$$

where

u = flow velocity

n = Manning's roughness value

CME = coefficient of 1 for metric units and 1.486 for English units

g = acceleration due to gravity

D = flow depth

21. Sand transport. The supply of sediment to and from the bed for non-cohesive bed material (sand) was controlled by the transport potential of the flow and the availability of material in the bed. The bed source term is

$$S = \frac{Ceq - C}{t_c} \tag{4}$$

where

S = source term

Ceq = equilibrium concentration

C = concentration of sediment

t_c = characteristic time for effecting the transition

Shoal study. The cohesive or clay version of STUDH was used in the Elizabeth River study.

The hydrodynamic model, RMA-2V

15. The hydrodynamic model, RMA-2V, solves the depth-integrated equations of conservation of mass and momentum in two horizontal directions. The present model is an improvement of an earlier version, RMA-2 (Norton and King 1977). The model is formulated in terms of velocities and turbulent exchange coefficients.

16. The finite element method using Galerkin weighted residuals is used to solve the conservation of mass and momentum equations. Individual elements may be either quadrilaterals or triangles and may have curved (parabolic) sides. The shape functions are quadratic for flow and linear for depth. Integration in space is Gaussian. Derivatives in time are replaced by nonlinear finite difference approximations.

17. The finite element solution is fully implicit and the set of simultaneous equations is solved by Newton-Raphson iteration. The solution is achieved using a front-type matrix inversion that assembles a portion of the matrix and solves that portion before assembling the next portion of the matrix. The front solver's efficiency is largely independent of bandwidth and thus does not require as much care in formation of the computational mesh as do traditional solvers. A detailed description of the model is given by McAnally et al. (1983).

The sediment transport model, STUDH

18. Convection-diffusion equation. The sediment transport model, STUDH, solves the depth-integrated convection-diffusion equation in two horizontal dimensions for a single sediment constituent. The basic convection-diffusion equation is presented in Ariathurai, MacArthur, and Krone (1977). The form of the solved equation is

$$\frac{\partial C}{\partial t} + u \frac{\partial C}{\partial x} + w \frac{\partial C}{\partial z} = \frac{\partial}{\partial x}\left(Dx \frac{\partial C}{\partial x}\right) + \frac{\partial}{\partial z}\left(Dz \frac{\partial C}{\partial z}\right) + \alpha_1 C + \alpha_2 \qquad (1)$$

where

C = concentration, kg/m^3

t = time, sec

u = flow velocity in x-direction, m/sec

x = primary flow direction, m

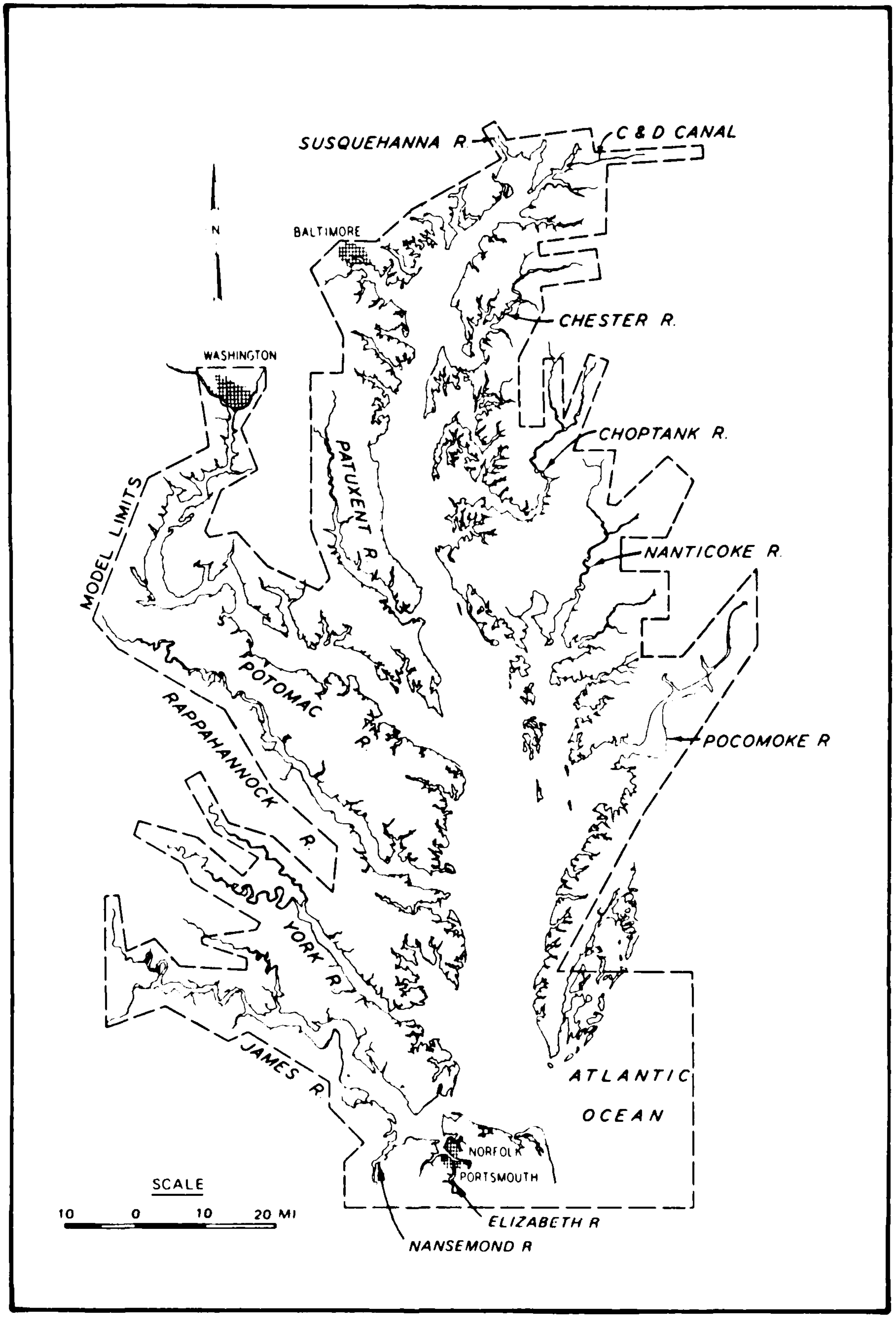

Figure 3. Physical model limits

proposed 50-ft channels leading into Baltimore and the existing channels elsewhere. Channels in the James and Elizabeth Rivers and Thimble Shoal area of the Lower Bay were molded to correspond to prototype information collected as late as 1981. The molded area of the model extends from approximately 30 miles offshore in the Atlantic Ocean to the heads of tide for all tributaries emptying into the Chesapeake. The entire length of the Chesapeake and Delaware (C&D) Canal and a portion of Delaware Bay are also molded. Overbank geometry is reproduced to the +20 ft contour. Model limits are shown in Figure 3.

12. The physical model was designed based on the equality of Froude numbers, model to prototype, reflecting similitude of gravitational effects. Geometric scales of the model are 1:1,000 horizontally and 1:100 vertically; the distortion ratio is 10:1. These dimensions and Froudian model laws defined the following model-to-prototype ratios:

Characteristic	Ratio
Vertical length	1:100
Horizontal length	1:1,000
Slope	10:1
Time	1:100
Velocity	1:10
Volume	1:100,000,000
Discharge	1:1,000,000

[illegible] -to-prototype ratio for salinity is 1:1. This is the general prac- [illegible] distorted-scale models.

[illegible] The model was designed and equipped so that selected prototype [illegible] conditions could be simulated and the model response to these condi- [illegible]. A detailed discussion of appurtenances necessary to generate [illegible] boundary conditions is provided in Report 1 of this series [illegible] Morton 1983).

The Numerical Models

[illegible] element hydrodynamic model, RMA-2V, was used in both the [illegible] study and the Elizabeth River study. The noncohesive or sand [illegible] finite element transport model, STUDH, was used in the Thimble

PART II: THE MODELS

The Hybrid Modeling Technique

9. The hybrid modeling technique used in this study combined experimental data from the Chesapeake Bay physical model with appropriate numerical models to provide an integrated method for predicting channel sedimentation. The resulting sedimentation predictions are considered to be more reliable than predictions that could be made using either physical or numerical models alone.

10. The numerical models used were a hydrodynamic model, called RMA-2V, and a sediment transport model, called STUDH. The Chesapeake Bay physical model provided the necessary water-surface elevation and current data to drive RMA-2V. The physical model information included both water-surface elevations and currents at all the RMA-2V boundaries and at locations within each RMA-2V [illegible] (Thimble Shoal and Elizabeth River). These locations within each model [illegible] used in either of two ways. One approach is to use these locations [illegible] internal boundaries, i.e., to drive each RMA-2V model just as physical model information drives the external water boundaries. In this way, the numerical hydrodynamic model is really used to interpolate the physical model [illegible] to include all RMA-2V node locations. Another approach is to drive [illegible] only at the external water boundaries and use the internal physical model information for comparison with RMA-2V results, thus allowing for a traditional calibration and verification of the numerical hydrodynamic model. Because of the relative sparseness of internal physical model data, the latter approach was taken for this study. The resulting currents at model locations [illegible] were then used by the numerical sediment transport model, STUDH. [illegible] discussion of the hybrid modeling approach is given by McAnally et [illegible].

Chesapeake Bay Physical Model

11. The physical model of Chesapeake Bay is located on Kent Island in [illegible], Maryland. The model is an 8.6-acre fixed-bed model molded in conformance with the most recent National Ocean Survey (NOS) charts. At [illegible] this study, all major ship channels had been molded with the

navigation channel using existing hybrid modeling techniques. The study was designed specifically to address the relative changes of both overall shoaling and distribution of shoaled material within the project limits caused by deepening.

Scope

8. The numerical modeling portion of the hybrid modeling effort consisted of two separate numerical models, which were referred to as the Thimble Shoal model and the Elizabeth River model. Two separate models were needed to encompass the study area rather than one overall model because of the varying nature of shoaled material along the project navigation channel. The sediment along the Thimble Shoal portion of the navigation channel consists predominantly of noncohesive material while the sediment along the Elizabeth River portion of the navigation project consists primarily of clays and silts (cohesive material). Thus the noncohesive version of the numerical sediment model was used for the Thimble Shoal study and the cohesive version of the numerical sediment model was used for the Elizabeth River study. Sedimentation in a third portion of the overall project, referred to as the Atlantic Ocean Channel, is not addressed in the main body of this report but is presented in Appendix A.

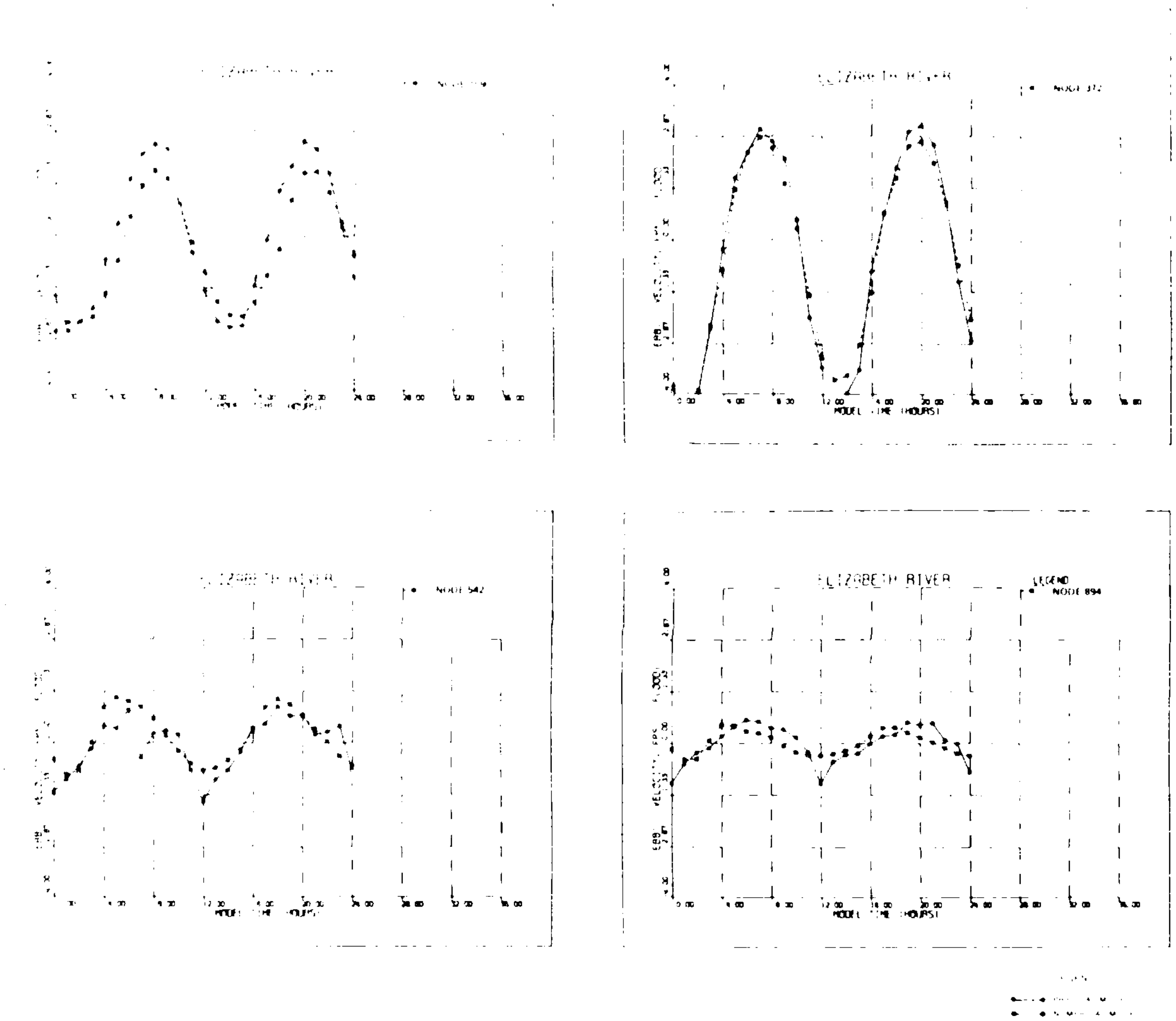

Figure 8. Elizabeth River velocity comparison (Base 1)

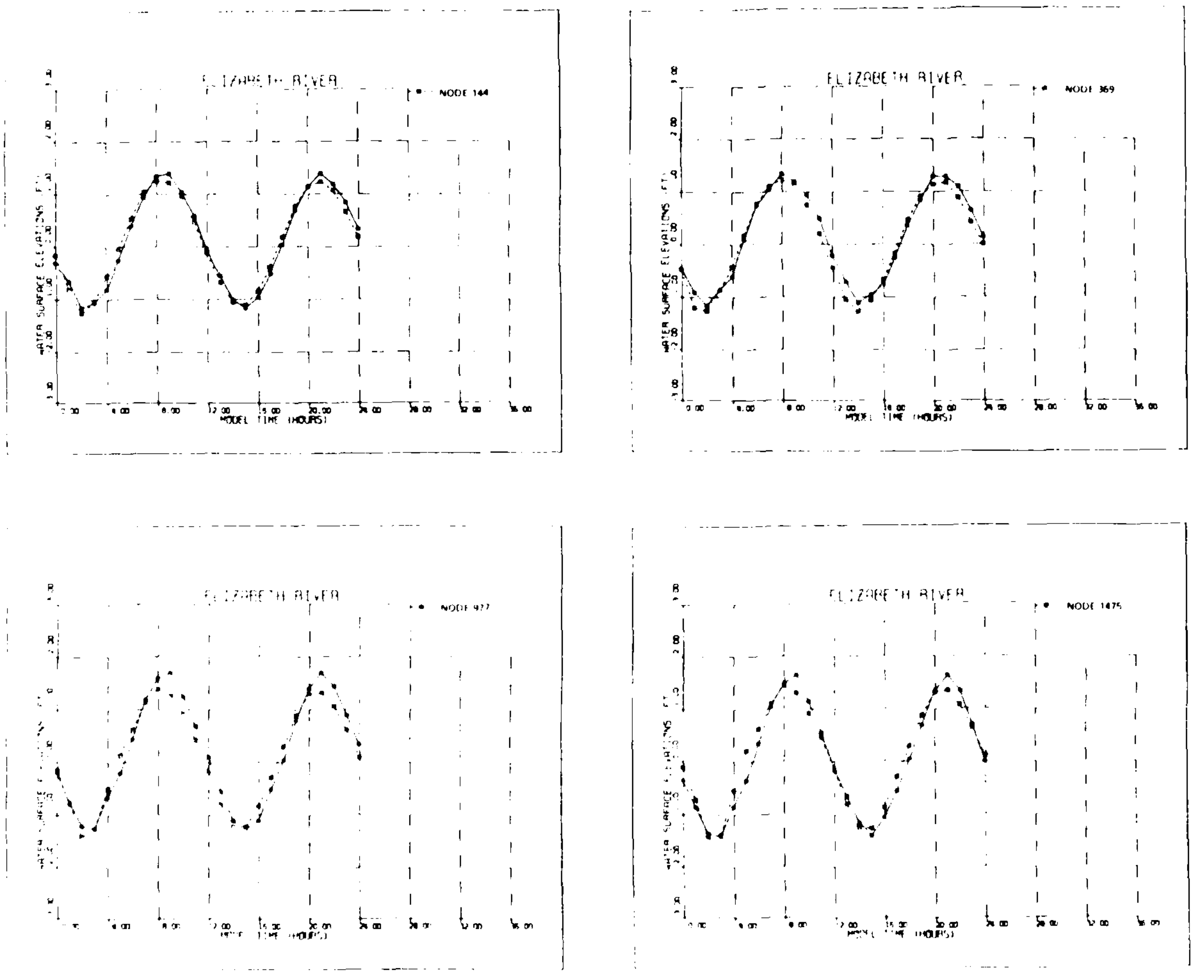

Figure 9. Elizabeth River water-surface elevation comparison (Base 4)

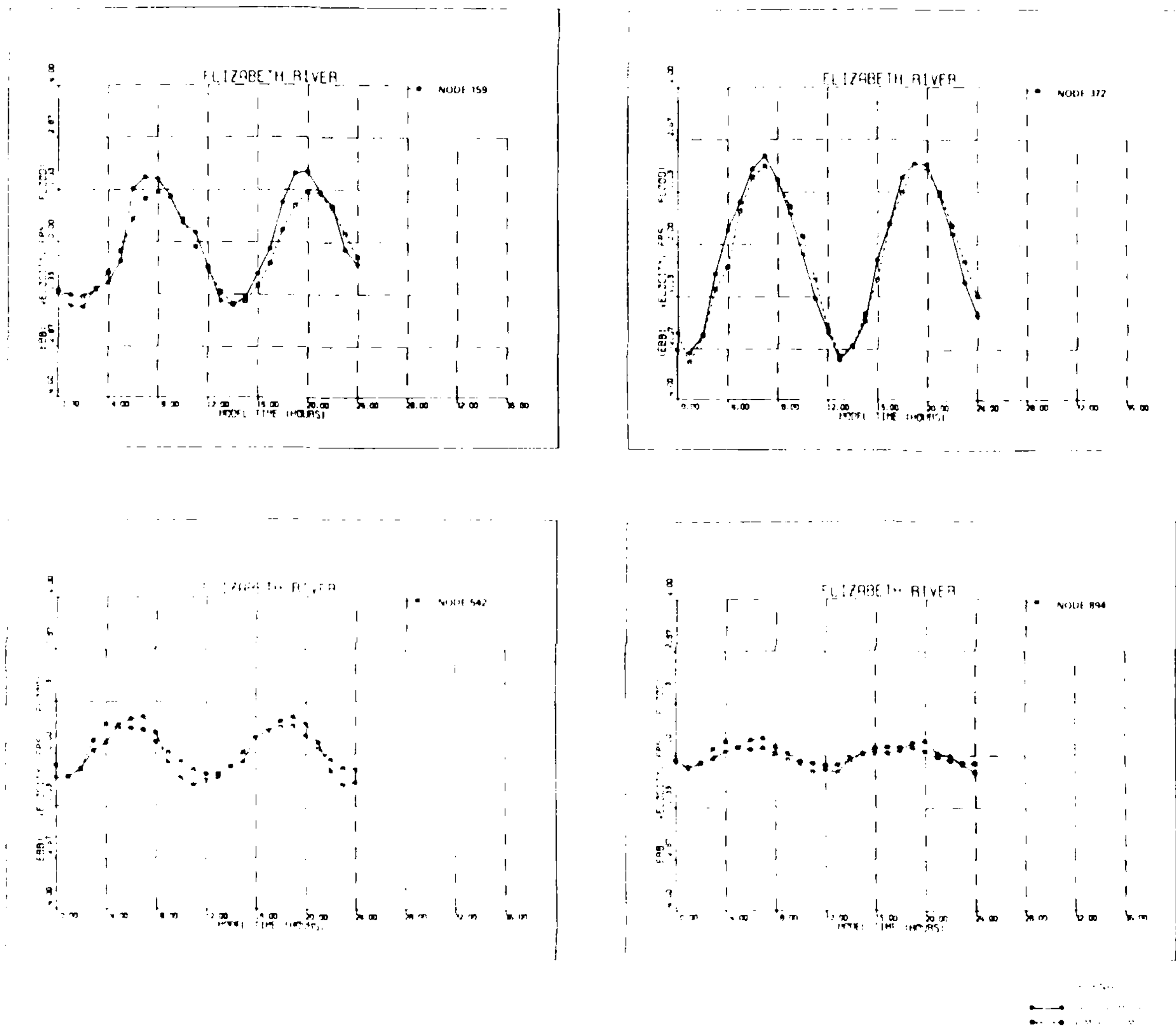

Figure 10. Elizabeth River velocity comparison (Base 4)

node for each time-step over a tidal cycle were then stored in a form that could be used by the sediment model.

33. The proposed deepened channel was simulated by deepening the channel areas to the project depth (including an additional 3 ft for maintenance and dredging tolerance) without changing any of the numerical model parameters. The deepened channel model was then run using the four sets of boundary conditions. Results of representative station velocity comparisons between the numerical model and the physical model are shown in Figure 11. The complete set of water-surface elevation and velocity comparisons for all test conditions are shown in Plates 1-24. The hydrodynamic model plan results were stored in the same manner as the base results for use in the sedimentation simulations.

Sedimentation simulation

34. In order to properly adjust the Elizabeth River sediment transport model, representative navigation channel shoaling rates had to be established from available prototype data. To that end, hydrographic surveys from 1962-1981 were acquired from the US Army Engineer District, Norfolk. The method used to compute navigation channel shoaling rates was to compare the post-dredge survey with the following predredge survey, thus establishing the amount of material that had shoaled during that period. For the existing 45-ft-deep channel reaches (Norfolk Harbor reach, Craney Island reach, and Lamberts Bend reach), five such comparisons could be made, all occurring during the time period from July 1976 to December 1981. For the existing 40-ft-deep channel reach (Port Norfolk reach, Town Point reach, and the Lower and Middle reaches of the Southern Branch), five comparisons could also be made, all occurring within the time period from August 1971 to February 1980. For the 35-ft-deep channel reach (the Upper reach of the Southern Branch), six comparisons could be made, all occurring within the period from May 1963 to February 1981.

35. The times used in establishing shoaling rates are as follows:

Channel	Period
45-ft project	(1) Apr 1981 to Dec 1981
	(2) Dec 1979 to Apr 1981
	(3) Oct 1978 to Dec 1979
	(4) Sep 1976 to Jun 1978
	(5) Mar 1976 to Aug 1976

(Continued)

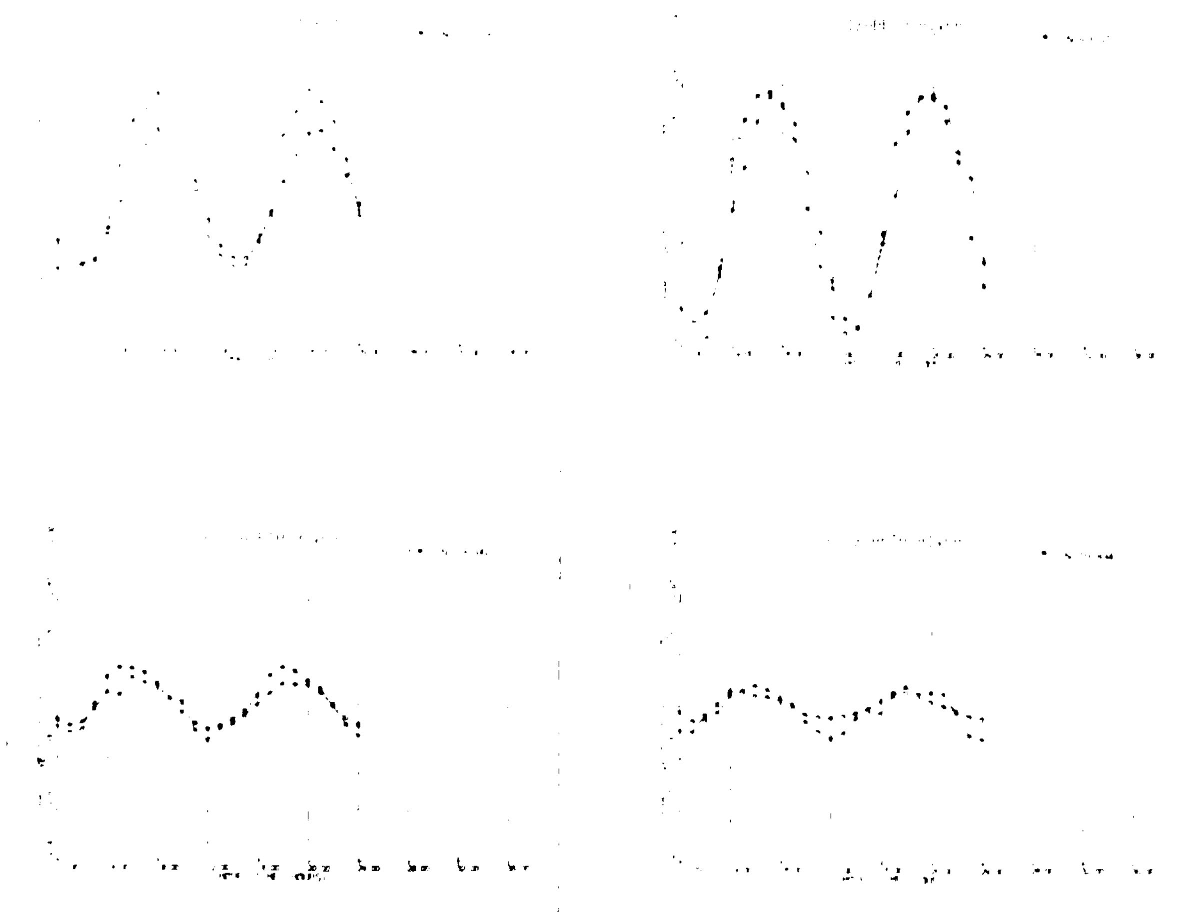

Figure 11. Elizabeth River numerical model-physical model velocity comparison (Plan 1)

Channel	Period
40-ft project	(1) Nov 1975 to Feb 1980
	(2) Jul 1974 to Sep 1975
	(3) Feb 1974 to Jul 1974
	(4) Mar 1973 to Sep 1973
	(5) Aug 1971 to Feb 1973
35-ft project	(1) Mar 1979 to Feb 1981
	(2) Apr 1978 to Mar 1979
	(3) Feb 1974 to Dec 1977
	(4) Jun 1972 to Sep 1973
	(5) Oct 1969 to Jun 1972
	(6) May 1963 to Oct 1969

36. To allow for comparison of prototype shoaling with the sediment transport model shoaling, the existing 45-ft-deep project was sectioned as shown in Figure 12; the 40-ft project was sectioned as shown in Figure 13; and the 35-ft project was sectioned as shown in Figure 14.

37. The prototype shoaling that occurred in each section for the periods evaluated and the averages used in model calibration are given in Table 1.

38. It is not yet economically feasible to run the hydrodynamic and sediment transport models for a continuous simulation of events for a year or even several months. Therefore, in order for the sediment transport model to simulate a long shoaling period, first several separate events were simulated independently. Then each event was extrapolated over the appropriate time interval and all were combined to simulate a representative annual sedimentation cycle. Four separate conditions were used for this type of simulation for the Norfolk Harbor study (identical conditions for both the Elizabeth River model and Thimble Shoal model). A description of these four events or conditions is given in paragraph 31. The impact of each condition on the long-term shoaling in the model was determined by subdividing the year of events into the portions of the time that could best be represented by a particular event.

39. The combination of two freshwater inflows and two tidal ranges yields four events or conditions. The tidal ranges at Old Point Comfort, which corresponded to the tidal range of the simulated events, were 2.40 and 3.70 ft, termed the mean and the high range events. The proportion of the simulated year in which the range was greater than 3.05 ft was considered the high range impact period; conversely, the time period proportion less than 3.05 ft was the mean.

40. These duration periods were ascertained from the tidal ranges

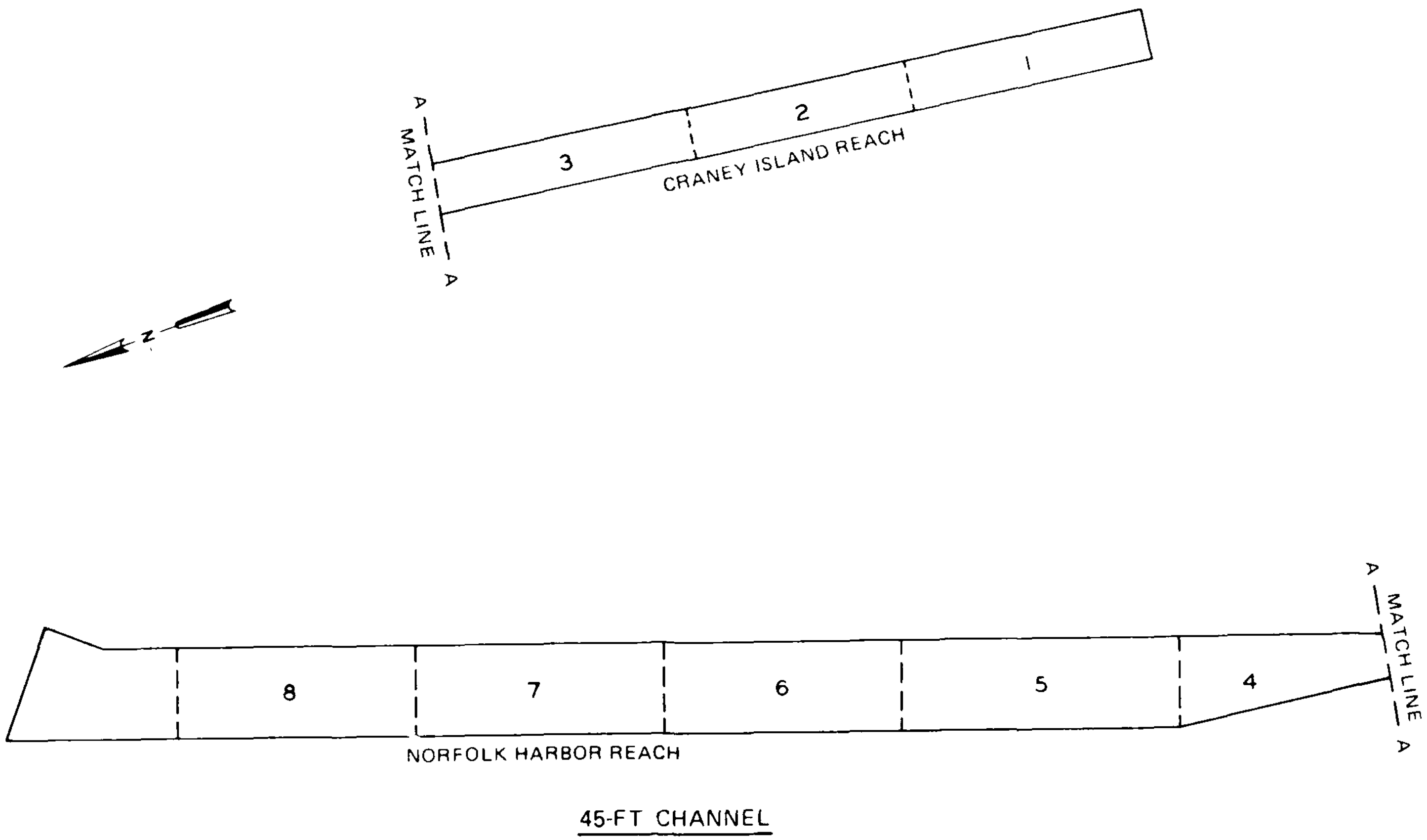

Figure 12. Elizabeth River 45-ft project-prototype shoaling sections

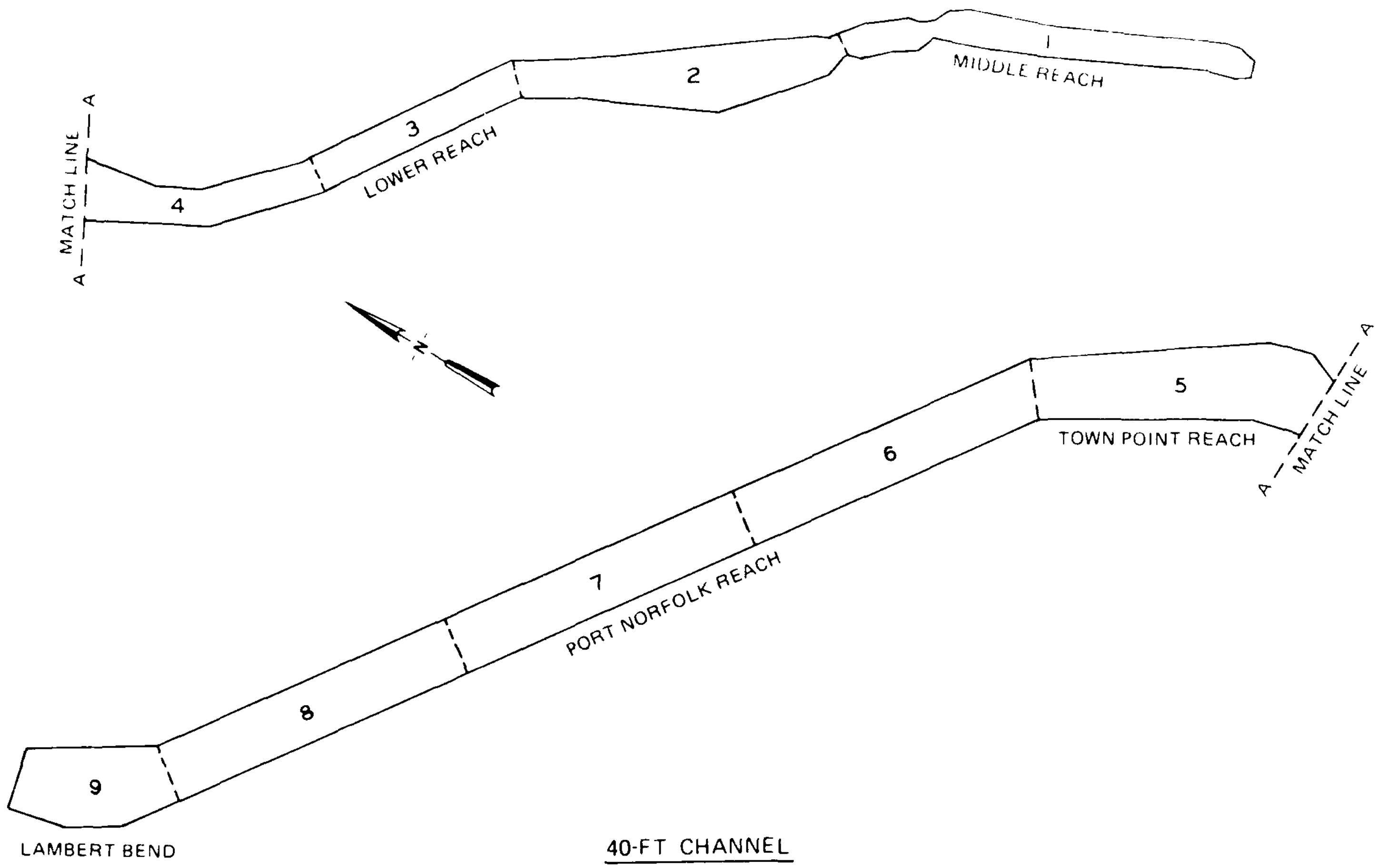

Figure 13. Elizabeth River 40-ft project-prototype shoaling sections

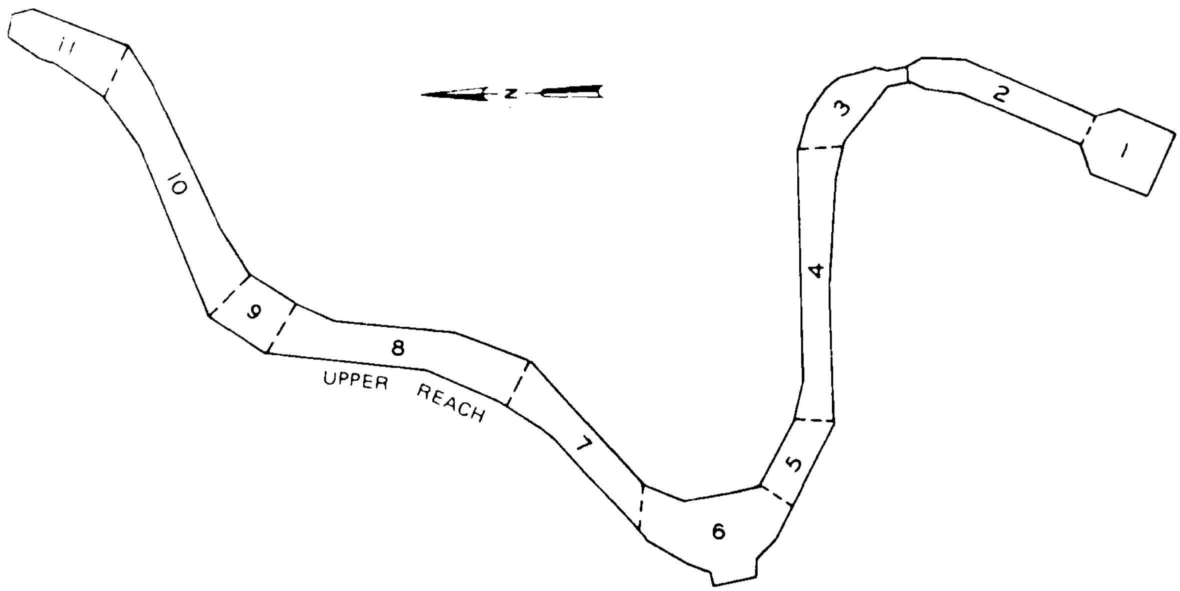

Figure 14. Elizabeth River 35-ft project-prototype shoaling sections

generated by the nine major astronomical tidal components at Old Point Comfort (Richards and Gulbrandsen 1982):

Constituent	Amplitude, ft	Period, hr
M2	1.188	12.421
N2	0.265	12.658
S2	0.230	12.000
K1	0.170	23.934
O1	0.146	25.819
V2	0.051	12.626
P1	0.048	24.066
L2	0.033	12.192
K2	0.059	11.967

Since actual tides were not used, meteorological effects were neglected; therefore the total energy may be slightly lower than actual values. At Old Point Comfort, the average astronomical tidal range was 2.44 ft; the mean simulation event had a range of 2.40 ft. The high range events would then be simulated 16 percent of the model year and the mean range events, 84 percent.

41. The inflow conditions in the physical model were:

	Mean Flow, cfs	High Flow, cfs
Nansemond River	372	1,063
Chickahominy River	289	826
Appomattox River	967	2,763
James River	7,249	20,711
Total inflow to Chesapeake Bay	70,000	200,000

42. The duration of flows in the James River near Richmond are from the Virginia Department of Water Resources (1974). From this reference, it was calculated that the high inflow event (greater than 13,980 cfs in the James River near Richmond) was exceeded only 10 percent of the time. Accordingly, the high flow events would be modeled for 10 percent of the time and the mean flow events the remaining 90 percent.

43. The combination of these independent inflow and range events would be:

Event	Range	Inflow	Duration Percent of Total
1	High	High	1.6
2	Mean	High	8.4
3	High	Mean	14.4
4	Mean	Mean	75.6

44. The various model parameters established during calibration were:

Time-step length	1,800 sec (0.5 hr)
Crank Nicholsen implicitness factor	0.70
Manning's n roughness	0.017
Critical shear stress for deposition	0.02 N/m^2
Critical shear stress for erosion	
Top layer	0.02 N/m^2
Second layer	0.06 N/m^2
Particle erosion constant	0.0012 $kg/m^2/sec$
Effective settling velocity	
James River	0.0008 m/sec
Within Elizabeth River	0.0001 m/sec
Dry weight density of deposit	500 kg/m^3

(Continued)

Sediment Concentration Boundary Conditions*		
	High Freshwater Inflow	Mean Freshwater Inflow
James River (upstream)	0.050 kg/m^3	0.035 kg/m^3
Chesapeake Bay (downstream)	0.010 kg/m^3	0.010 kg/m^3
Upper Elizabeth River	0.001 kg/m^3	0.001 kg/m^3

* Values based on Onishi and Wise (1978).

45. The length of the time-step was chosen to not allow sediment to pass completely through an element during one time-step. Many of the other parameters are present to address the cohesive sediment transport processes of this region. Sediment within the Elizabeth River was largely silts and clays and so was cohesive in character. The critical shear stresses for deposition and erosion were chosen to allow deposition and resuspension (erosion) in the Norfolk Harbor reach and primarily deposition alone throughout most of the Elizabeth River.

46. A model-to-prototype comparison, model calibration, is shown in Figure 15. These model shoaling rates were the superposition of the influence of each of the four flow events.

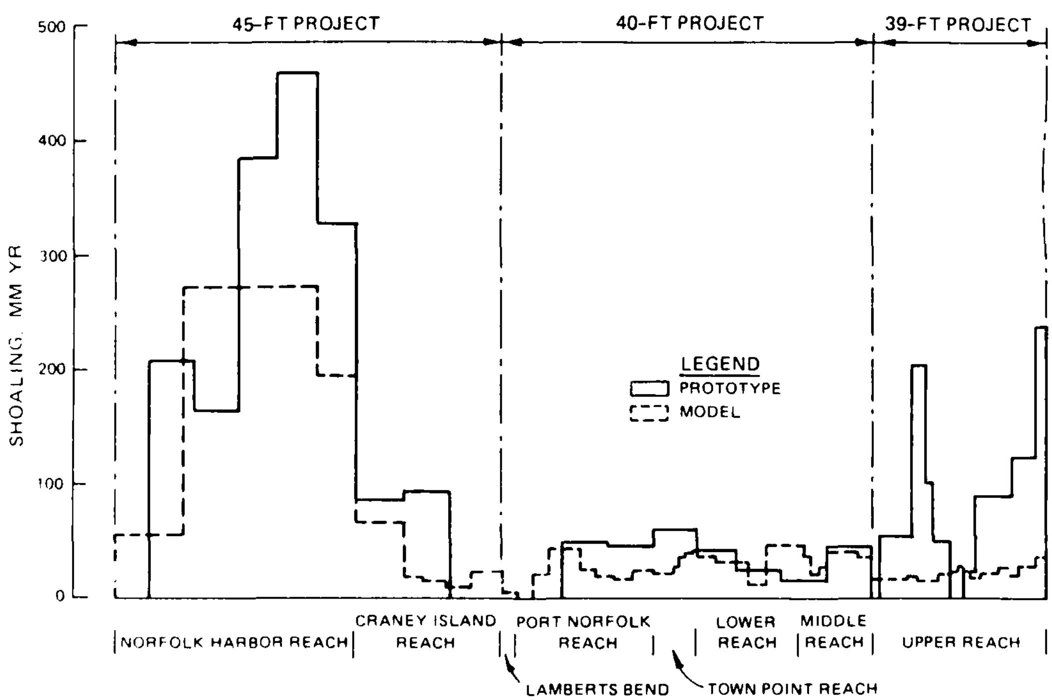

Figure 15. Elizabeth River model sedimentation verification

47. The model's maximum shoaling, about 270 mm/yr (prototype maximum of 460 mm/yr) occurred in the same area of the Norfolk Harbor reach as in the prototype. The model and prototype shoaling rates dropped within the Elizabeth River channels. The Craney Island disposal area effectively blocked material from entering the Elizabeth River mouth from the west; also the currents are somewhat stronger due to the smaller cross section. Both prototype and model results indicated a minimum deposition around Lamberts Bend.

48. In the Port Norfolk reach, the model showed a peak shoaling rate of about 45 mm/yr; this shoaling rate showed a decline from there to the upper reach of the project. The prototype data showed shoaling rates in Elizabeth River with greater variation than those in the model, and with a peak in the turning basin at the head of the upper reach. The model sediment source was derived mostly from the James River, while the prototype might have had a local source in addition. However, the overall model shoaling pattern was quite similar to that of prototype and the magnitudes were quite reasonable.

Thimble Shoal

Hydrodynamic simulation

49. The same procedures used in the Elizabeth River simulations were used in the Thimble Shoal simulations. The boundary file source was still the physical model data and all station comparisons were made to the physical model stations as in the Elizabeth River simulations.

50. NOS chart numbers 12248 and 12222 were used as the basis for the mesh overlay representation of the lower portion of Chesapeake Bay and the entrance to the James River estuary. The Thimble Shoal mesh constructed consisted of 1,395 nodes and 428 elements. Figure 16 shows the resulting mesh. The upstream and downstream water boundary conditions of the Thimble Shoal mesh were velocity or discharge at Hampton Roads, the entrance to the Elizabeth River, and the northern cutoff of the Chesapeake Bay and tidal elevations at the ocean.

51. The same type of physical model data furnished for the Elizabeth River simulations was also used for the Thimble Shoal simulations. All the test conditions remained the same, only the computational mesh and boundary locations were different. Figures 17 and 18 show the locations of the comparative physical model stations with numerical model nodes.

Figure 16. Thimble Shoal mesh

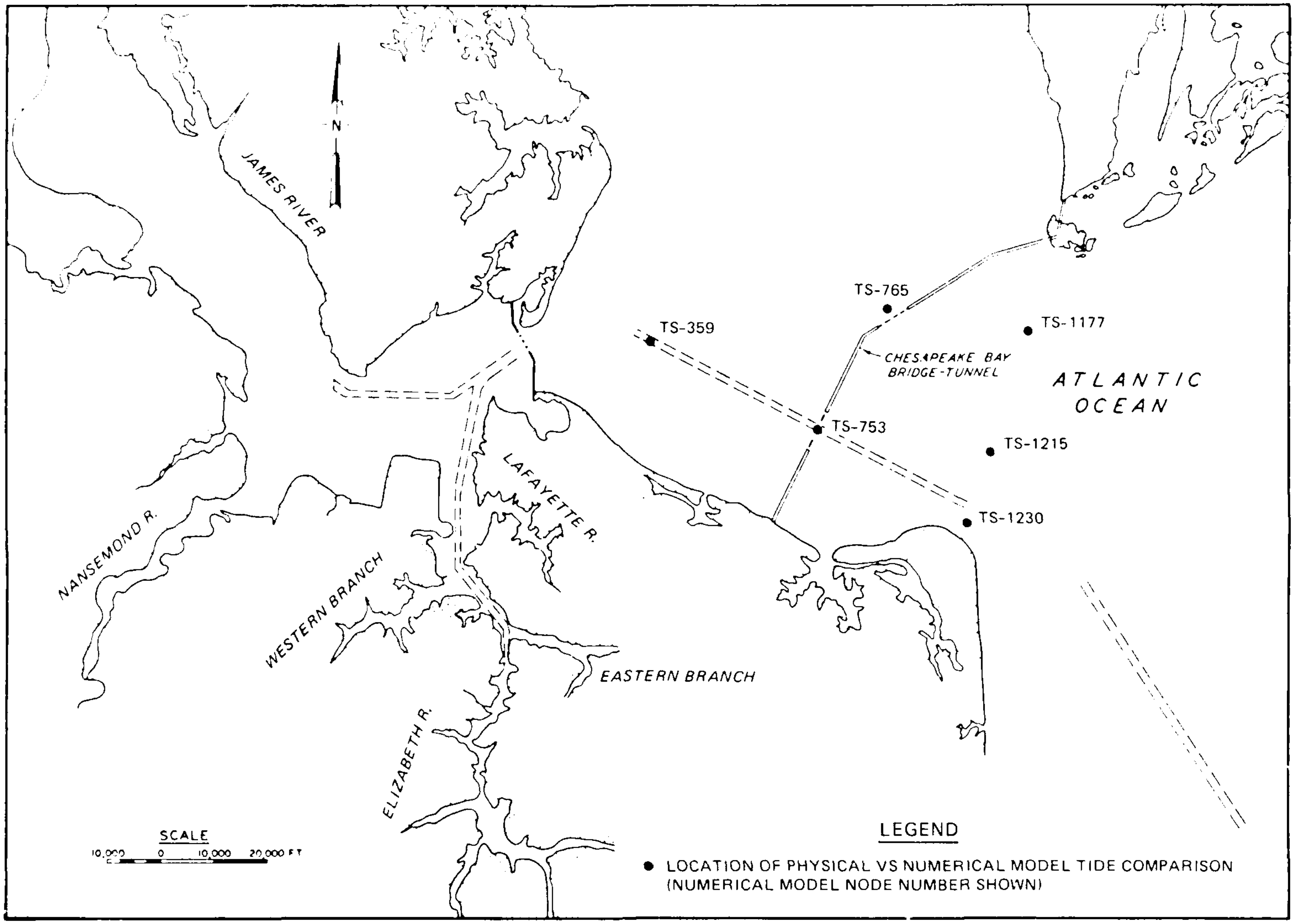

Figure 17. Location and node number of numerical model nodes used for comparison with physical model tide stations, Thimble Shoal (TS)

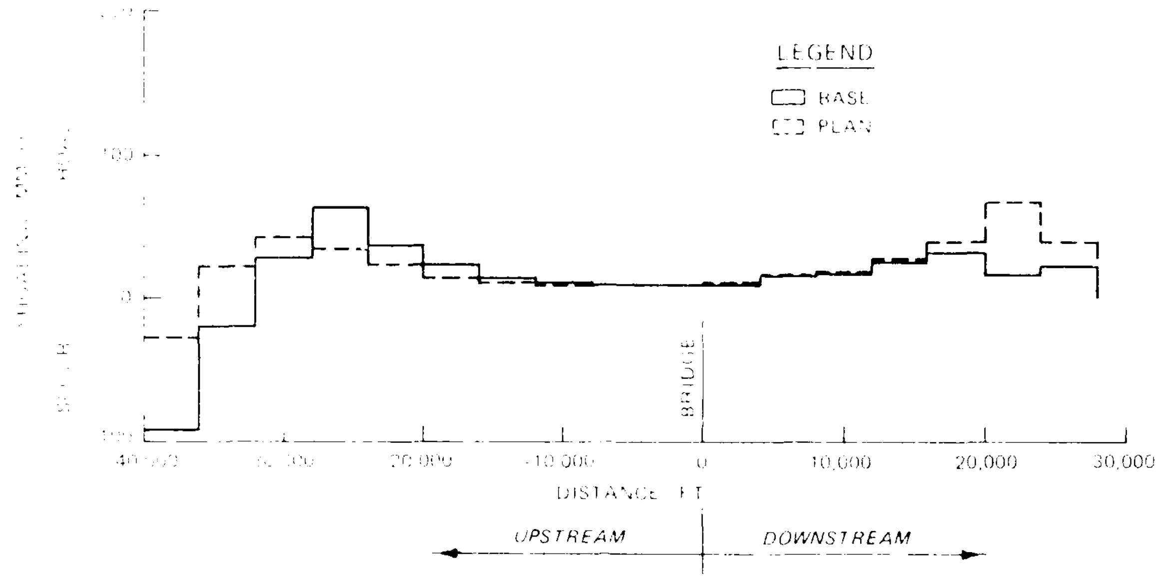

Figure 31. Thimble Shoal, sedimentation results,
annual rate comparison

section between ranges -36,000 to -40,000 ft was reduced from about 90 to 30 mm/yr. It is possible that the lack of this material source is responsible for the apparent shoaling reduction in this region.

70. The amount of shoaling represented an annual volume increase of about 20 percent from base to deepened plan conditions. If one considered the net sedimentation rates (scouring and shoaling), the plan demonstrated a 65 percent increase over base, the first determination (20 percent) would be the appropriate manner to calculate the potential increase maintenance dredging volume, as areas that showed net scour would not need to be dredged.

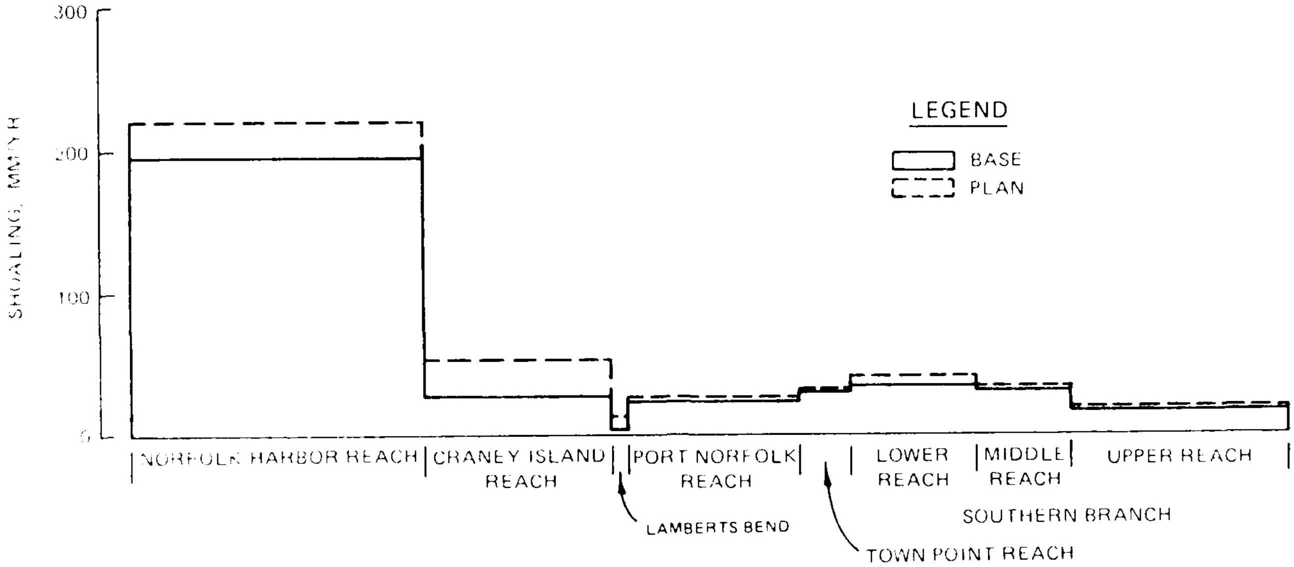

Figure 30. Elizabeth River, sedimentation results, annual rate comparison

deposition, each of the four conditions for Thimble Shoal consisted of both scour and deposition. The conditions are related in that deposition in one condition is erosion in another condition, making results of a single condition less meaningful than in the Elizabeth River simulation. For this reason only, the yearly base versus plan comparison is shown (Figure 30) for Thimble Shoal. The channel deepening had its greatest impact on the lower four reaches (Norfolk Harbor, Craney Island, Lamberts Bend, and Port Norfolk reaches). Velocities in this region were large enough to resuspend some of the freshly deposited material; therefore the deepenings reduction of velocity here had a greater effect. The annual volume of channel deposition was 23 percent higher over the length of the channel for plan relative to base.

Thimble Shoal Channel

68. Comparison of model results between base and after deepening (plan) conditions is shown in Figure 31. The shoaling peaks, both west and east of the bridge, moved outward toward the ends of the channel. This would seem to indicate that the material depositing entered from each end, and with the deepened conditions the current velocities were not capable of carrying the material as far into the channel.

69. The seaward peak was increased from about 30 to 70 mm/yr. The peak landward shoaling rate was reduced from about 65 to 45 mm/yr. Scour in the

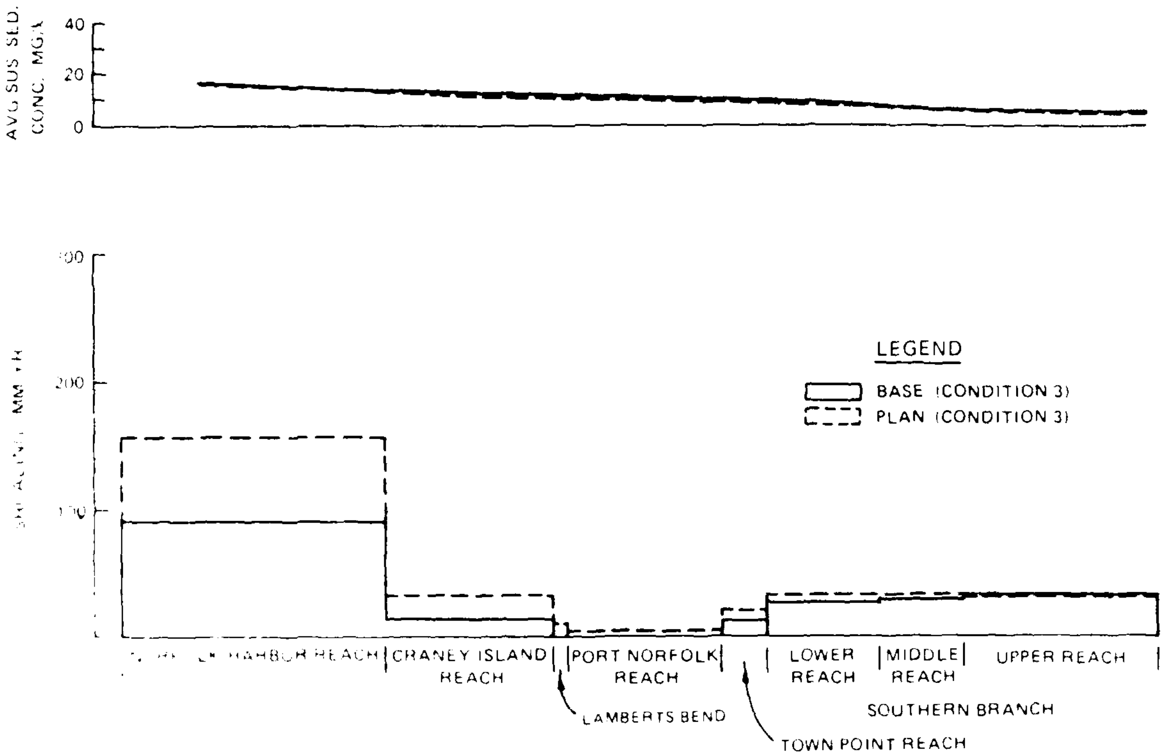

Figure 28. Elizabeth River, sedimentation results, base to plan (Flow 3)

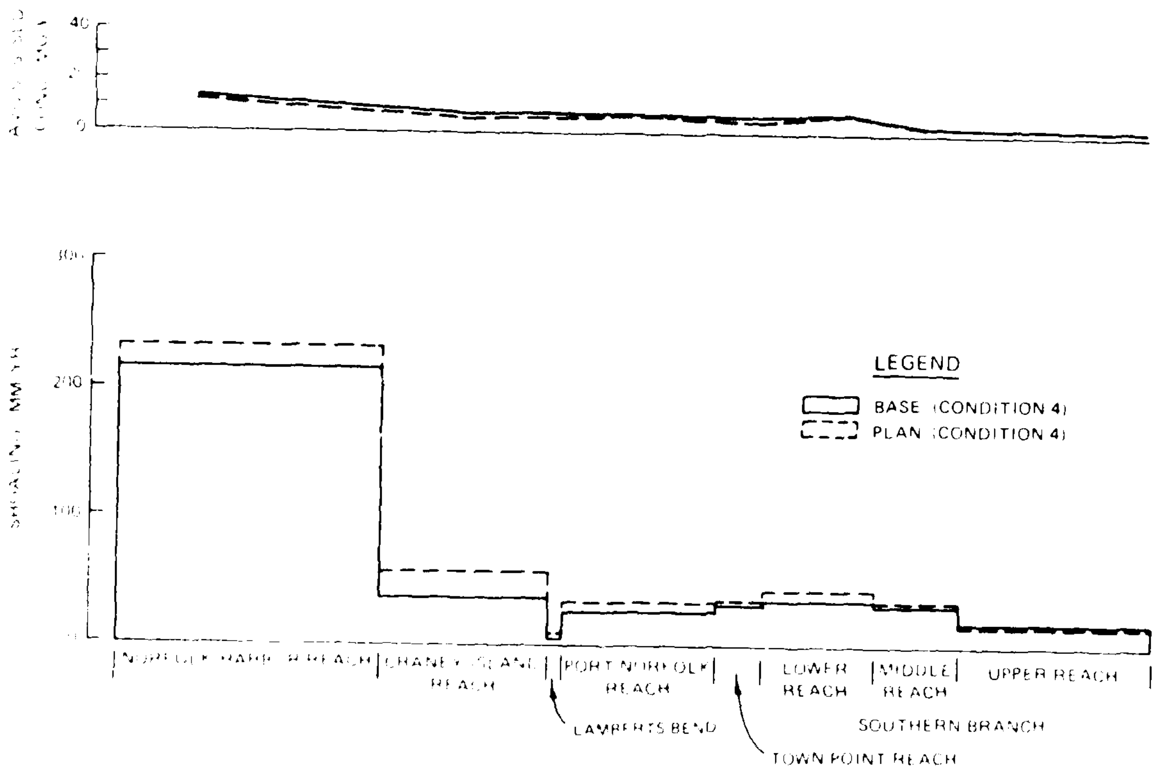

Figure 29. Elizabeth River, sedimentation results, base to plan (Flow 4)

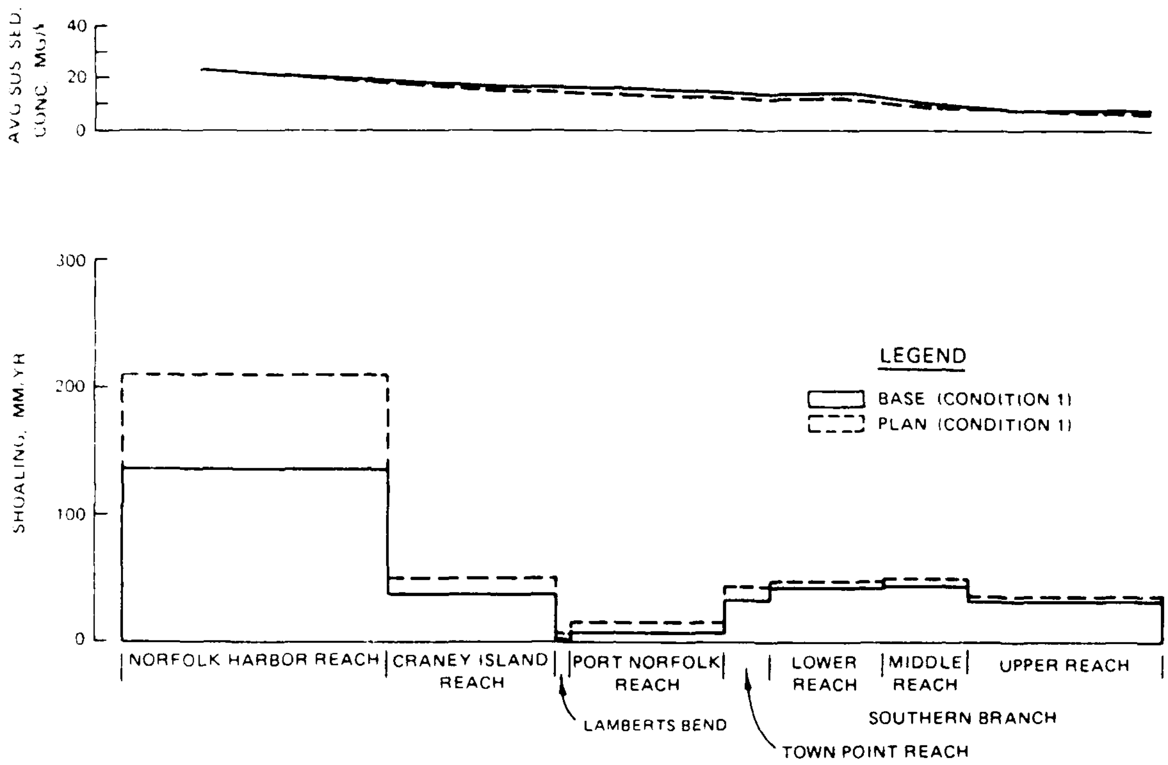

Figure 26. Elizabeth River, sedimentation results, base to plan (Flow 1)

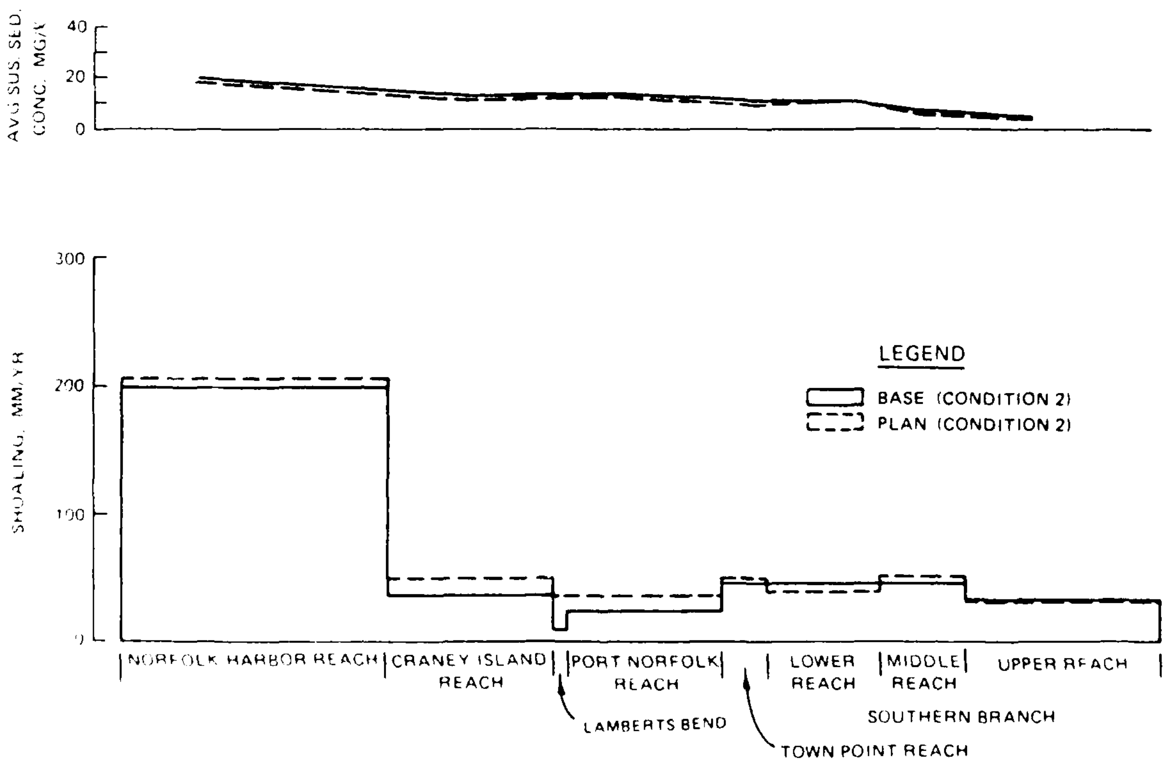

Figure 27. Elizabeth River, sedimentation results, base to plan (Flow 2)

PART IV: SEDIMENTATION RESULTS

Elizabeth River

64. Comparisons of existing (base) and deepened (plan) channel sedimentation results for each flow event are shown in Figures 26-29. These figures were derived on an annual basis for each event, so the magnitudes have only relative significance. Each figure also includes the average suspended sediment concentration profile along the channel. The suspended sediment concentration profile indicated a slight reduction in the channel for the plan for all four events. There is an increase in shoaling rate in most reaches of the Elizabeth River under all four conditions of the new deepened channel.

65. The rate of sedimentation was highest in the Norfolk Harbor reach for all four events for both plan and base conditions. However, the high range events (Events 1 and 3) shoaled substantially less in this reach than did the lower range events (Events 2 and 4). The greater tidal prism of the Elizabeth River produced higher flow magnitudes and thus was able to reduce deposition. The difference between the plan and base shoaling rates was greatest for the high range events. The increased channel depth reduced current velocity magnitudes and subsequently the bed shear stress and therefore did not allow as much resuspension of freshly deposited material. For the lower tidal range events, the bed shear stress did not generate a great deal of resuspension in base and so the increased channel depth did not increase net deposition very much.

66. Another interesting observation is that the high flow events (Events 1 and 2) shoaled somewhat less than the lower flow event. It should be noted that the events termed "low" for tidal range and freshwater inflow are in fact "mean" events. It would be a mistake to place a great deal of emphasis on such a detailed analysis of model results without substantiation from the prototype. Since the model was calibrated to a large time period, combining a multitude of events, its validity would be limited to that degree of comparison.

67. Figure 30 shows the comparison utilizing a combination of the four events for an annual estimate of shoaling for base and plan conditions by channel reaches. The base versus plan comparisons for each of the four conditions are not displayed. Unlike the Elizabeth River results which were mainly

this particular region. The purpose of the study, to determine the maintenance dredging change due to channel deepening, should still be addressable since deepening of the channel should not result in the channel trapping any more material from the migratory shoal.

62. The model results showed the same shoaling pattern as that of prototype but were generally lower in magnitude.

63. Shoaling outside the channel domain was not noticeable in the sediment transport model; this seems to be in line with a previous estimate of 3.7 mm/yr in this region (Ludwick 1981).

took place upstream of the dredged navigation channel around the narrows at the James River mouth.

60. The model calibration results from the combination of the three simulated events are shown in Figure 25. The largest prototype shoaling rate occurred between ranges -20,000 and -24,000 ft and is about 100 mm/yr. The sedimentation model showed a peak between ranges -24,000 and -28,000 ft and was about 65 mm/yr. The model shoaling peak here is due to the ebb plume from the James River. Another interesting feature is the net scour indicated by the surveys in the segment from range -36,000 to -40,000 ft of about 60 mm/yr. This may be an indication that the system is still adjusting for the channel presence or perhaps the location of the natural channel is shifting. The model indicated a scour of about 90 mm/yr at this segment.

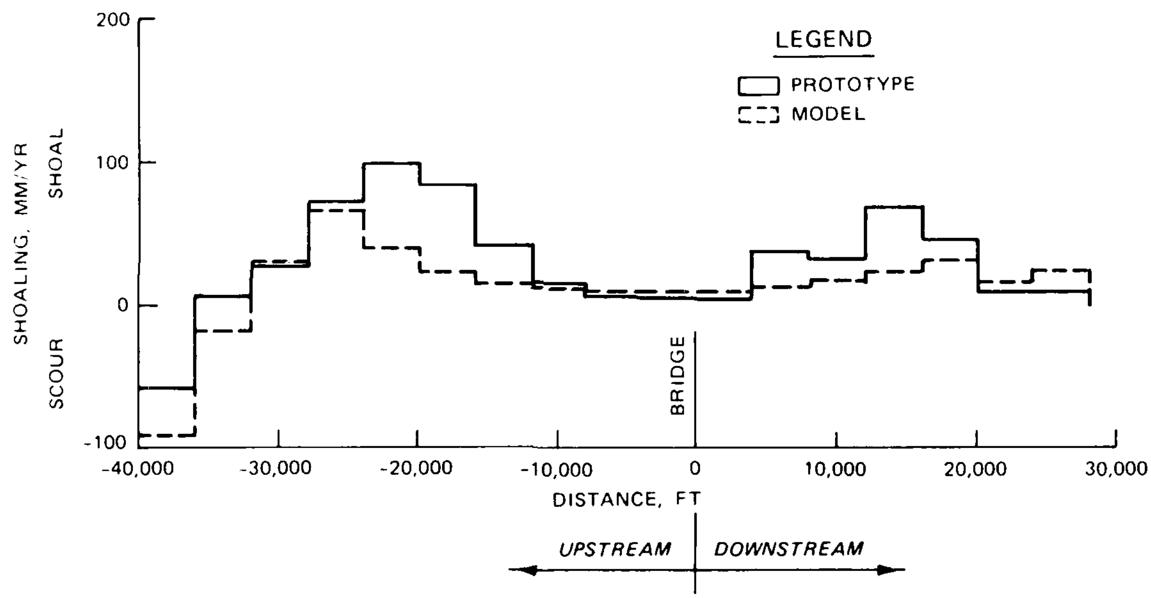

Figure 25. Thimble Shoal sedimentation verification

61. The second shoaling peak in the prototype (between ranges +12,000 and +16,000 ft) was about 60 mm/yr. This shoal was in the near vicinity of the "Tail of the Horseshoe" shoal, a mobile shoal just north of the channel which according to the findings of Ludwick (1981) has "experienced net accretion on its southern flank during the past century." This could be an explanation for the second peak. The model showed a less distinct peak than the prototype; this occurred between ranges +16,000 and +20,000 ft and was about 30 mm/yr. The sedimentation in the model is low here in spite of using an effective sediment diameter for transport smaller than was generally found in

Two other periods of shorter duration (less than 1 year) were available for analysis; but due to the very low shoaling rates in the Thimble Shoal Channel and the limited accuracy of hydrographic surveys, the decision was made to omit these short duration comparisons and use only the 1975-1981 period for calibration of the model.

55. To allow for comparison of prototype shoaling with the sediment transport model shoaling, the Thimble Shoal Channel was sectioned as shown in Figure 24.

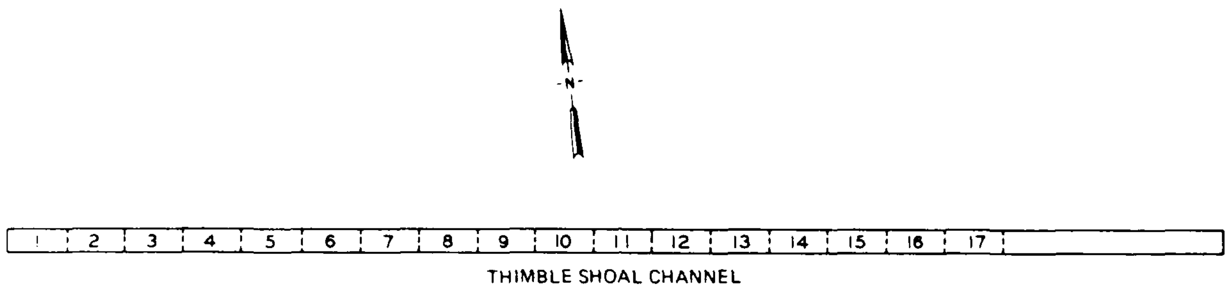

Figure 24. Thimble Shoal Channel-prototype shoaling sections

56. The prototype shoaling that occurred in each section for the period evaluated is given in Table 2.

57. The simulation of an annual event was conducted in the same manner as described in paragraph 38. However, difficulties arose in the hydrodynamic simulation of Event 3. Therefore Event 1 was used for both high range events.

58. The length of the sedimentation model time-step selected was 1,800 sec (0.5 hr). This was small enough not to allow advection of sediment completely through an element during one time-step. The computational grid was identical with that of the hydrodynamic model. Other model parameters were generally as follows:

Crank Nicholsen implicitness factor	0.65
Manning's n roughness	0.017
Effective particle diameter for transport	0.05 mm
Effective settling velocity	0.0008 m/sec
Boundary concentration	0.010 kg/m^3

59. This modeling effort was conducted assuming noncohesive sediment transport with the given effective particle diameter for transport as being coarse silt to very fine sand. This transport size appears most applicable in the western portion of the navigation channel (refer to Figure 4 from Ludwick (1981)). Even using this fairly small diameter, most of the sediment movement

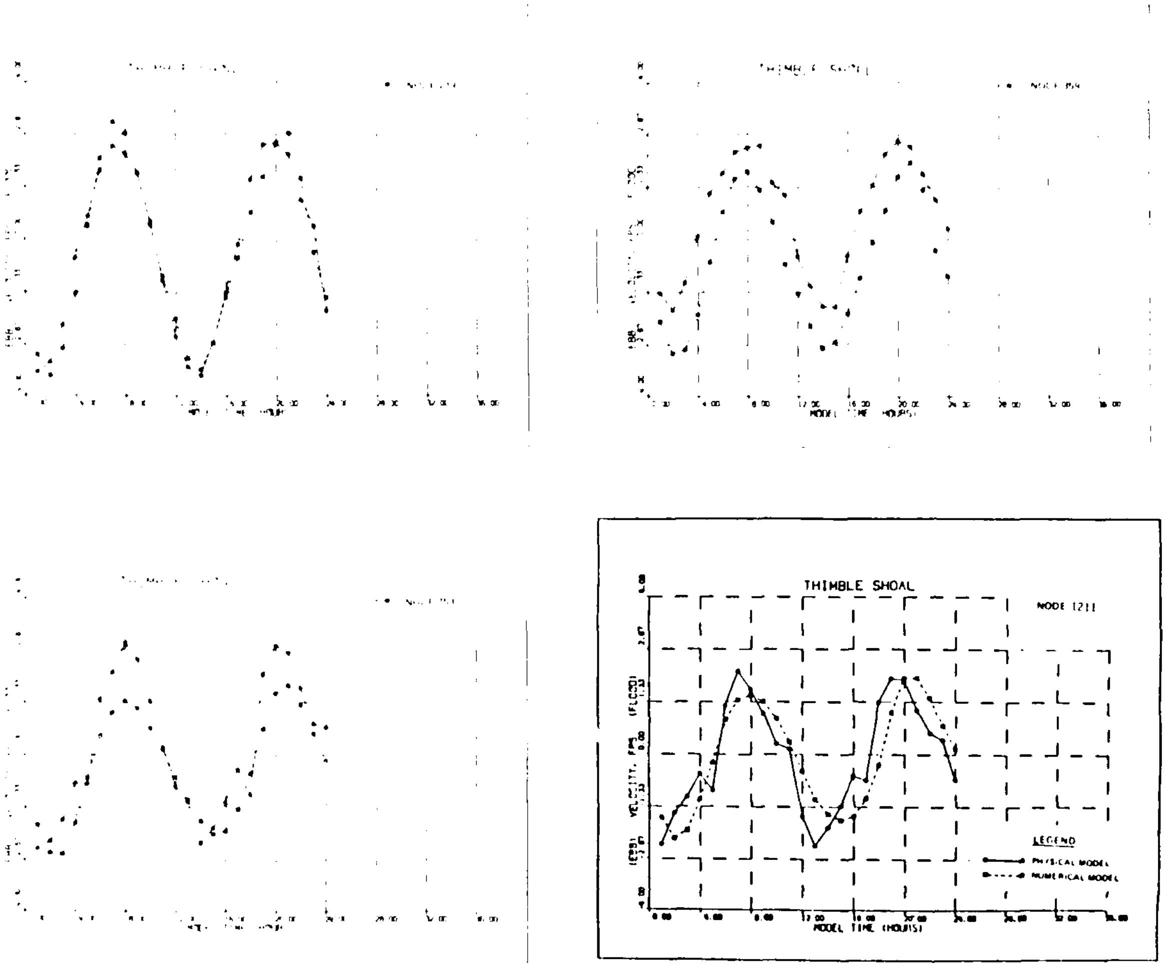

Figure 23. Thimble Shoal numerical model-physical model velocity comparison (Plan 1)

model and the physical model are shown in Figure 23. Water-surface elevation and velocity comparisons for all test conditions can be found in Plates 25-48. After verification was achieved, the hydrodynamic model results were stored for use in the sedimentation simulations.

<u>Sediment simulation</u>

54. In order to properly adjust the Thimble Shoal sediment transport model, representative navigation channel shoaling rates had to be established from available prototype data. Hydrographic surveys from 1960-1981 were acquired from the Norfolk District. The method used to compute navigation channel shoaling was the same as that described in paragraph 34 for the Elizabeth River model. The single period from 10 February 1975 to 14 April 1981 served as the calibration standard for the sediment transport model.

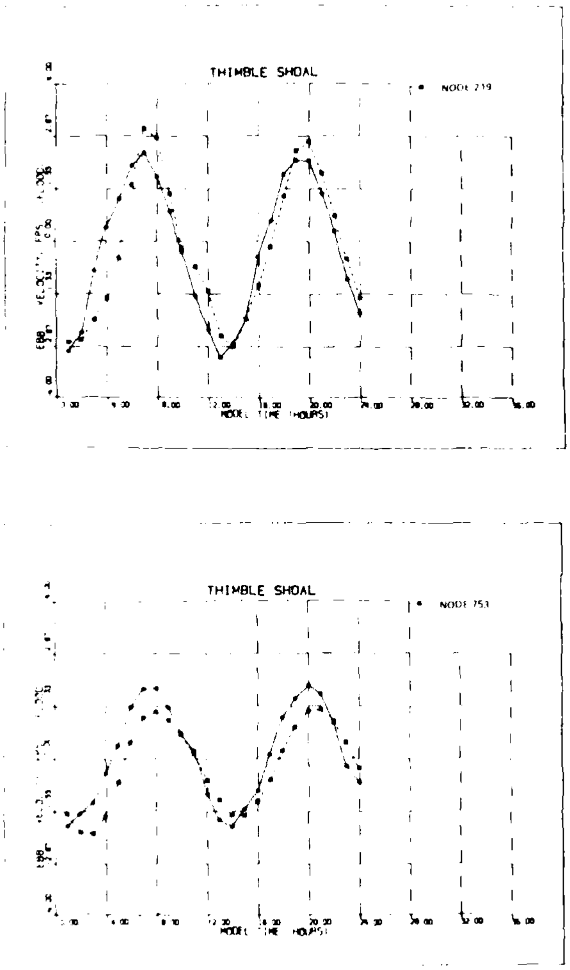

Figure 22. Thimble Shoal velocity comparisons (Base 4)

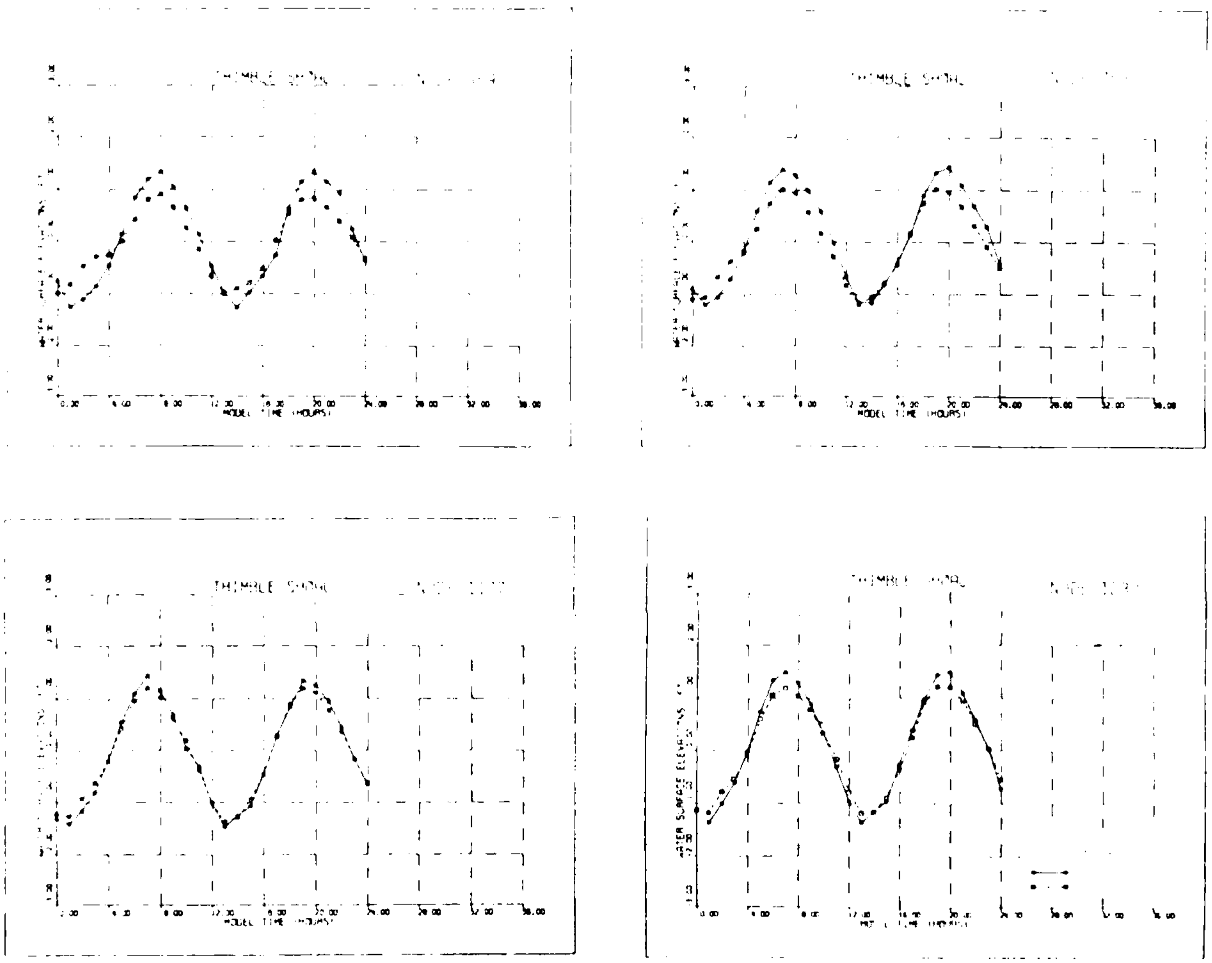

Figure 21. Thimble Shoal water-surface elevation comparison (Base 4)

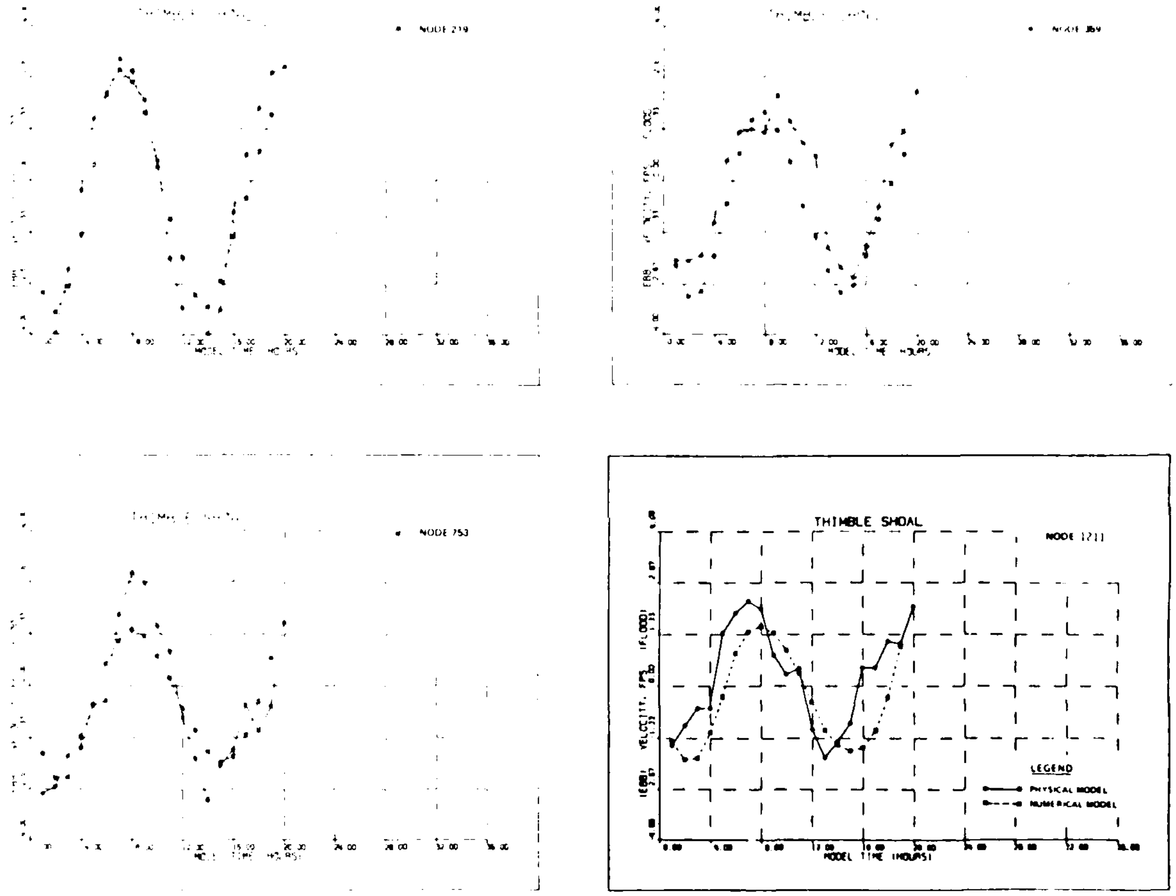

Figure 20. Thimble Shoal velocity comparison (Base 1)

52. Calibration of the numerical hydrodynamic simulation proceeded in the same fashion as that of the Elizabeth River simulation. The friction or eddy diffusivity parameters were adjusted until satisfactory agreement was achieved between the numerical and physical models. Then the numerical model was verified against Base 1 conditions. Finally, Base 2 and 3 conditions were run to complete the series. Figures 19-22 show tidal and velocity comparisons for critical representative locations. The hydrodynamic results were then stored for use in the sedimentation simulations.

53. The nodes representing the Thimble Shoal Channel were deepened to a uniform depth of 58 ft. This plan depth included the project depth and 3 ft for dredging tolerance and advance maintenance. The same boundary conditions used for the base simulations were then used for the deepened channel configuration. Results of a representative station comparison between the numerical

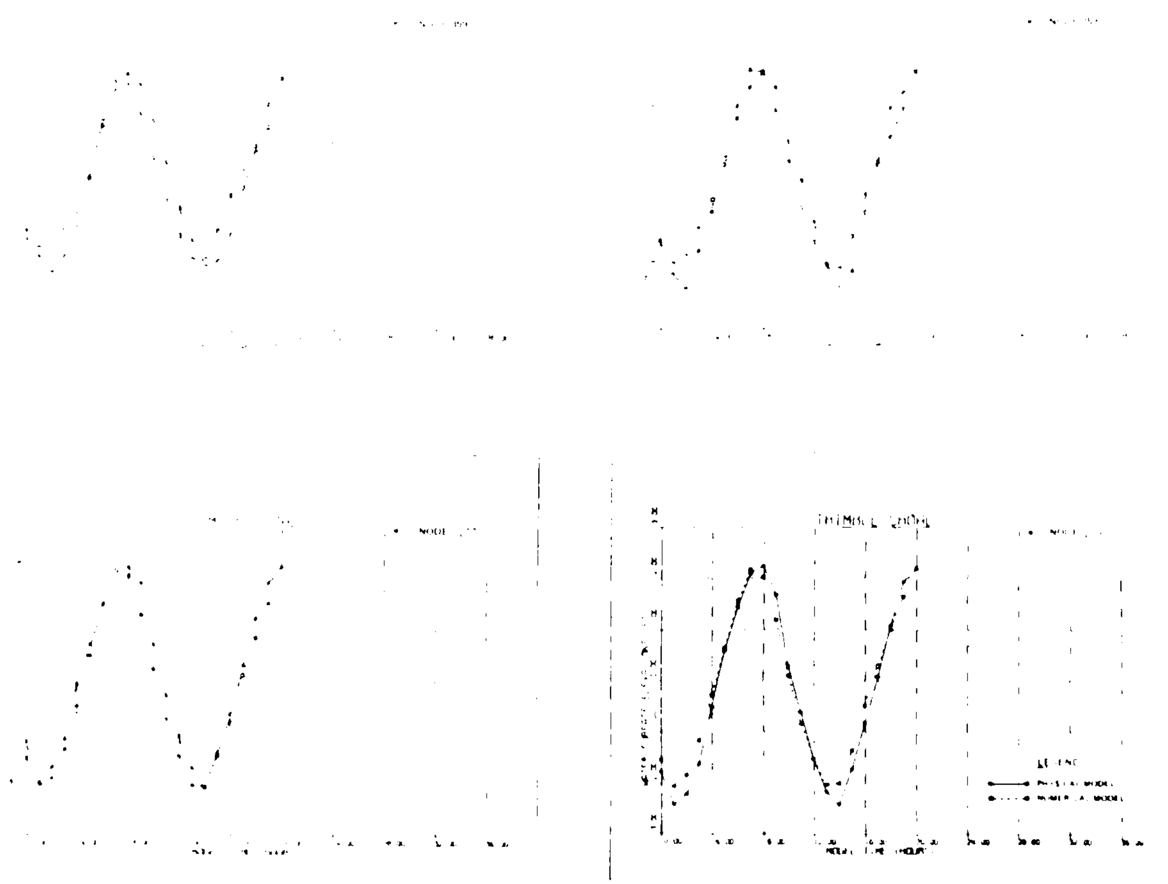

Figure 19. Thimble Shoal water-surface elevation comparison (Base 1)

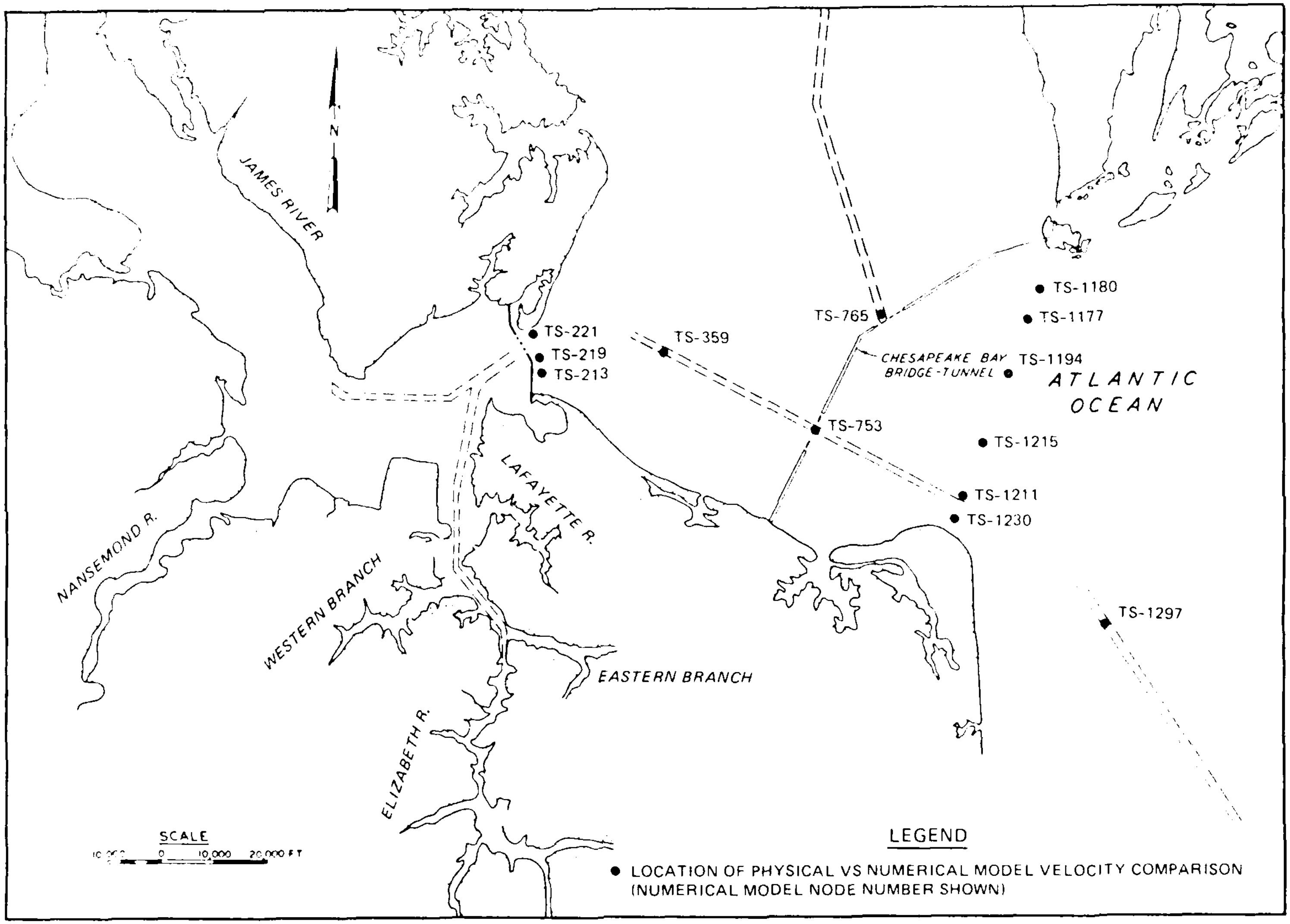

Figure 18. Location and node number of numerical model nodes used for comparison with physical model velocity stations, Thimble Shoal (TS)

PART V: CONCLUSIONS

71. Based on sedimentation results from the Elizabeth River numerical model, the increase in annual shoaling caused by channel deepening as proposed will be 23 percent. The distribution of shoaled material will not be significantly altered, other than a slight increase in skewness toward the downstream end.

72. Based on sedimentation results from the Thimble Shoal numerical model, the increase in shoaling caused by channel deepening as proposed will be about 20 percent. The distribution of shoaled material will be slightly altered in that both the upper and lower channel shoaling peaks which presently exist will tend to migrate even more toward the ends of the dredged channel.

73. The estimate of shoaling for the new Atlantic Ocean Channel, described in the appendix to this report, is about 200,000 cu yd annually.

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Table 1

Elizabeth River Channel Sedimentation, Infill Rate in Feet per Year

Period No.	Section 1	2	3	4	5	6	7	8	9	10	11
35-ft Project											
1	0.87	0.38	0.17	0.25	-0.39	-0.16	0.05	-0.31	-0.12	0.46	0.23
2	-0.46	0.60	-0.72	-0.38	-0.47	-0.23	0.38	0.67	1.84	-0.24	-1.67
3	0.55	0.03	0.16	0.07	-0.69	-0.47	-0.04	0.24	0.49	0.02	0.01
4	1.94	0.62	1.00	0.82	1.30	0.32	0.02	0.67	0.32	0.11	0.30
5	0.39	0.30	0.41	0.65	0.37	0.40	0.53	0.36	0.70	0.28	0.14
6	0.97	0.51	0.39	0.23	0.39	0.02	0.16	0.49	1.07	0.22	--
Avg*	0.78	0.38	0.29	0.29	0.07	-0.05	0.16	0.32	0.67	0.18	-0.03
40-ft Project											
1	0.02	-0.09	0.19	0.08	0.11	0.10	0.14	0.03	-0.23		
2	-0.24	0.27	-1.27	-0.25	-0.36	0.41	0.08	-0.26	0.15		
3	2.92	1.70	2.43	2.42	2.47	0.12	0.78	-0.43	--		
4	0.53	0.02	-0.21	-0.37	0.32	1.49	0.85	0.67	--		
5	-0.09	-0.21	0.28	0.12	0.22	-0.63	-0.32	-0.44	--		
Avg*	0.15	0.06	0.08	0.14	0.20	0.15	0.16	-0.05	-0.13		
45-ft Project											
1	-2.18	-1.39	0.31	1.57	1.08	1.61	0.30	1.03			
2	0.94	1.34	0.32	0.91	1.83	1.19	0.58	0.45			
3	--	--	--	1.09	1.49	1.64	0.44	0.86			
4	--	--	--	1.16	1.53	0.97	0.79	0.66			
5	--	--	0.10	0.38	1.33	0.89	0.06	0.03			
Avg*	-0.27	0.30	0.28	1.07	1.51	1.26	0.54	0.69			

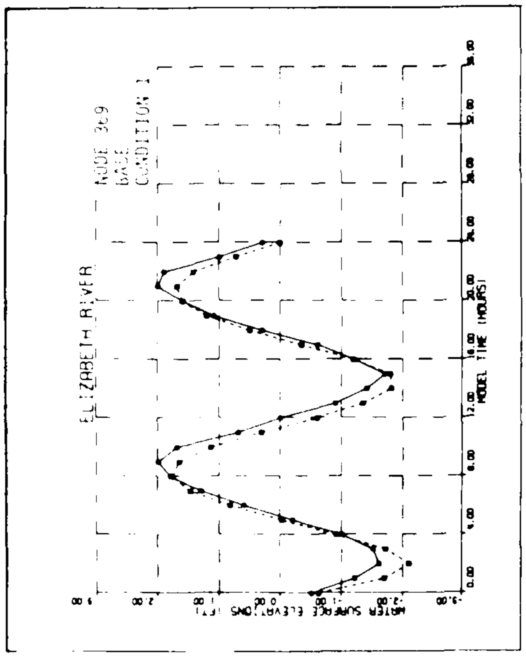
* Time weighted based on length of each shoaling period.

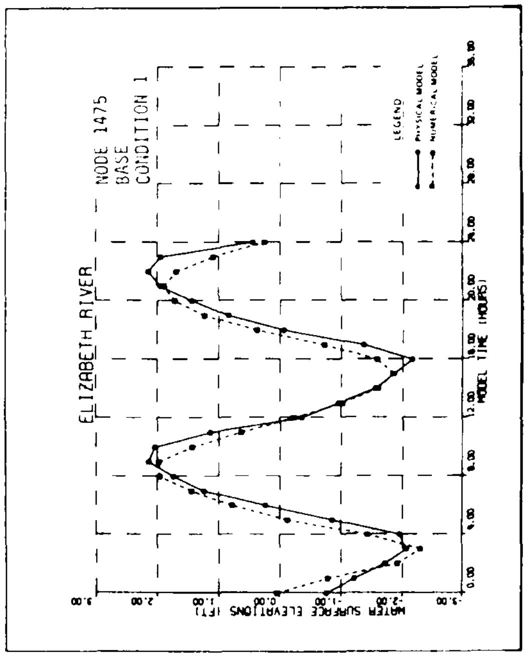
Table 2

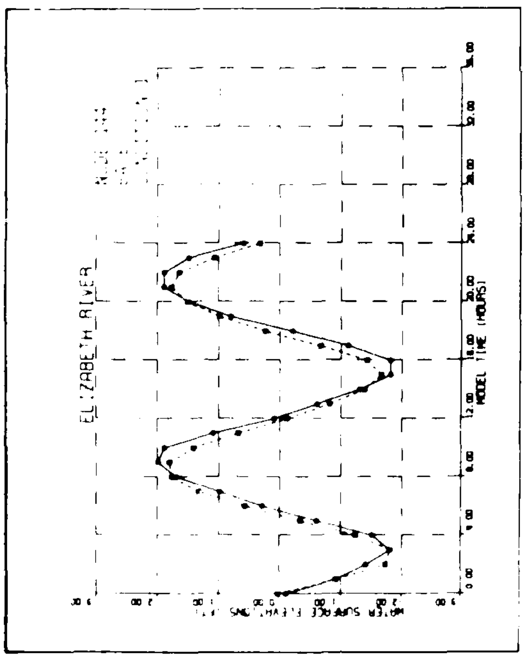
Thimble Shoal Channel Sedimentation from

10 February 1975 to 14 April 1981

Section	Infill Rate ft/yr
1	-0.20
2	0.03
3	0.11
4	0.25
5	0.33
6	0.26
7	0.15
8	0.14
9	0.05
10	0.03
11	0.02
12	0.13
13	0.12
14	0.24
15	0.16
16	0.03
17	--


ELIZABETH RIVER
BASE
CONDITION 1
WATER SURFACE ELEVATIONS (FT)
MODEL TIME (HOURS)


ELIZABETH RIVER
NODE 1475
BASE
CONDITION 1
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL
WATER SURFACE ELEVATIONS (FT)
MODEL TIME (HOURS)


ELIZABETH RIVER
MODEL TIME (HOURS)

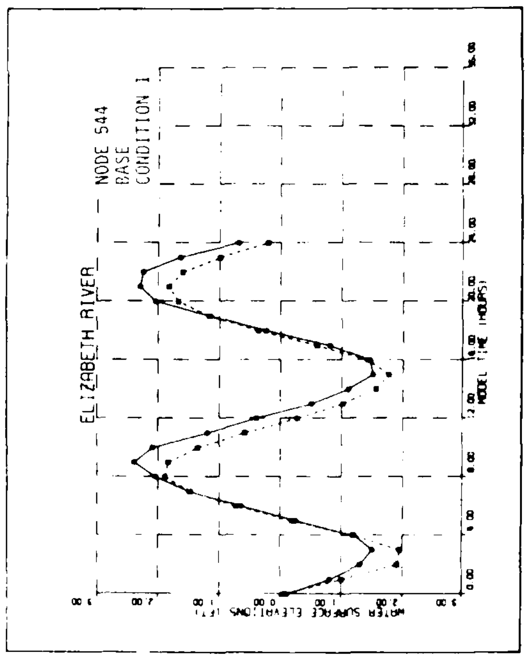

ELIZABETH RIVER
NODE 544
BASE
CONDITION 1
WATER SURFACE ELEVATIONS (FT)
MODEL TIME (HOURS)

PLATE 1

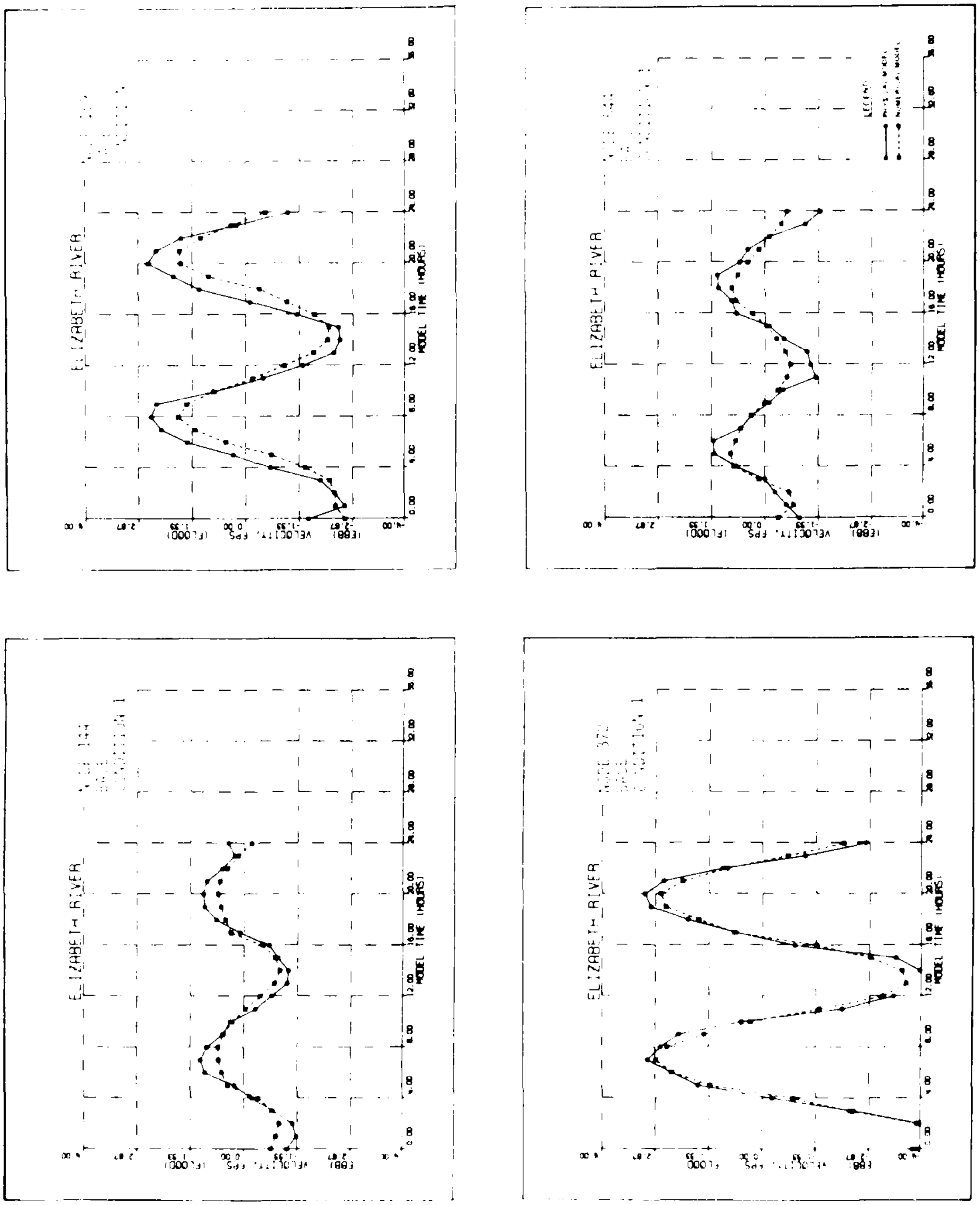
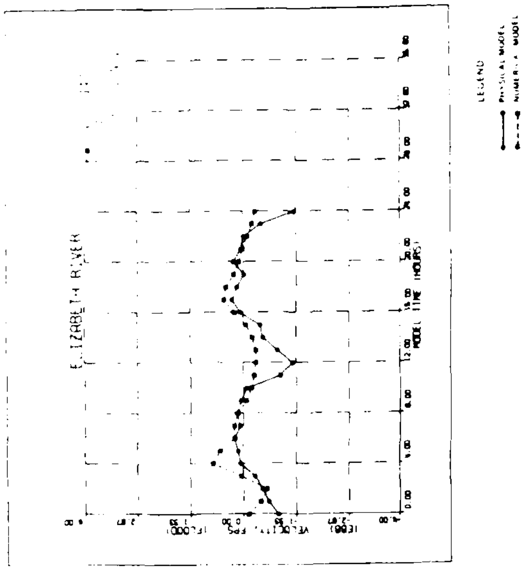

ELIZABETH RIVER
MODEL TIME (HOURS)
VELOCITY, FPS
(FLOOD)
(EBB)
LEGEND

PLATE 2


ELIZABETH RIVER
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)
LEGEND

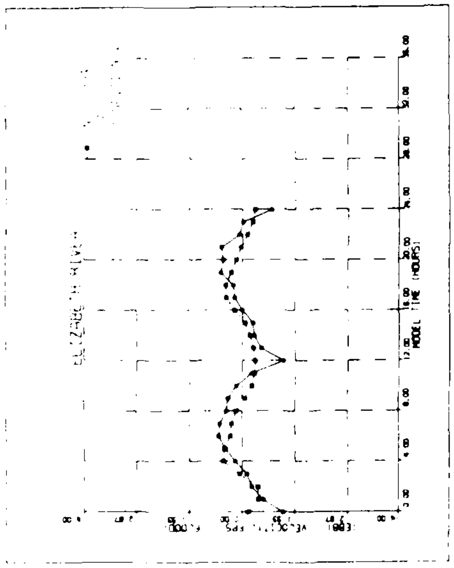
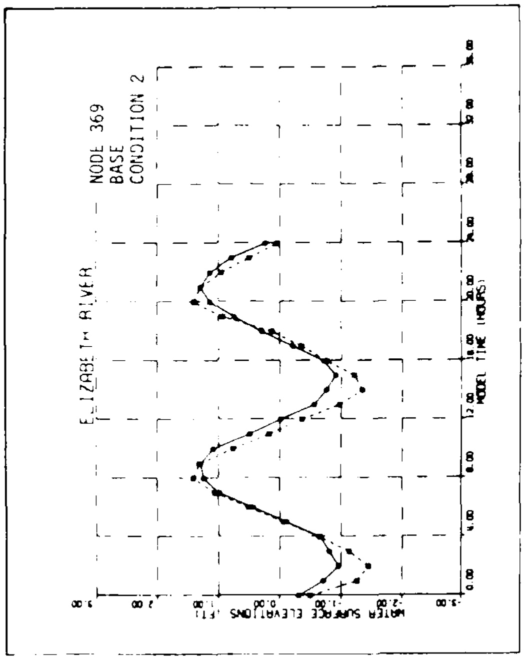
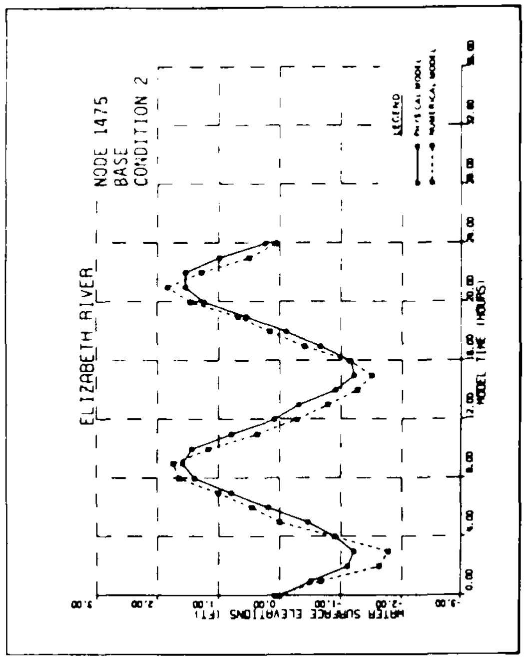
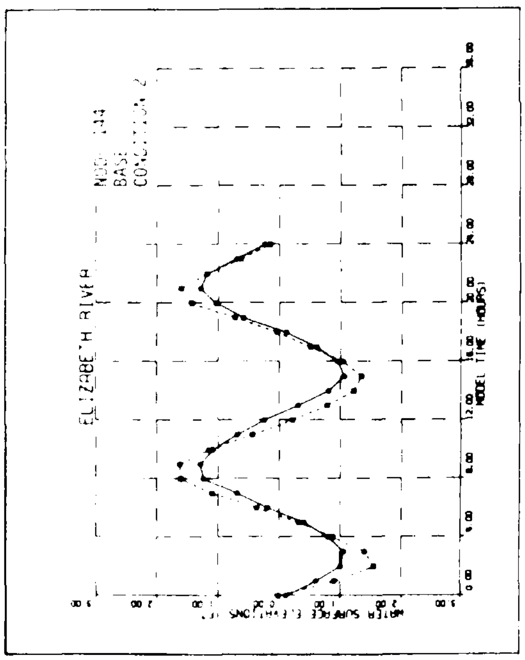

ELIZABETH RIVER
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)

PLATE 3


ELIZABETH RIVER
NODE 369
BASE
CONDITION 2
MODEL TIME (HOURS)
WATER SURFACE ELEVATIONS (FT)


ELIZABETH RIVER
NODE 1475
BASE
CONDITION 2
LEGEND
MODEL TIME (HOURS)
WATER SURFACE ELEVATIONS (FT)


ELIZABETH RIVER
BASE
MODEL TIME (HOURS)

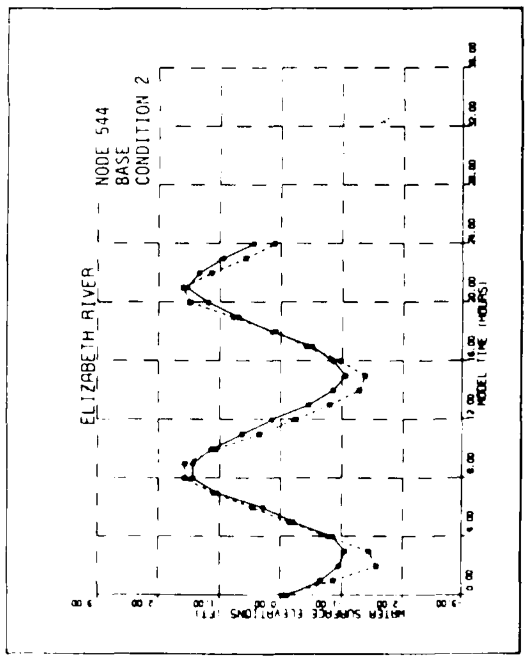

ELIZABETH RIVER
NODE 544
BASE
CONDITION 2
MODEL TIME (HOURS)
WATER SURFACE ELEVATIONS (FT)

PLATE 4

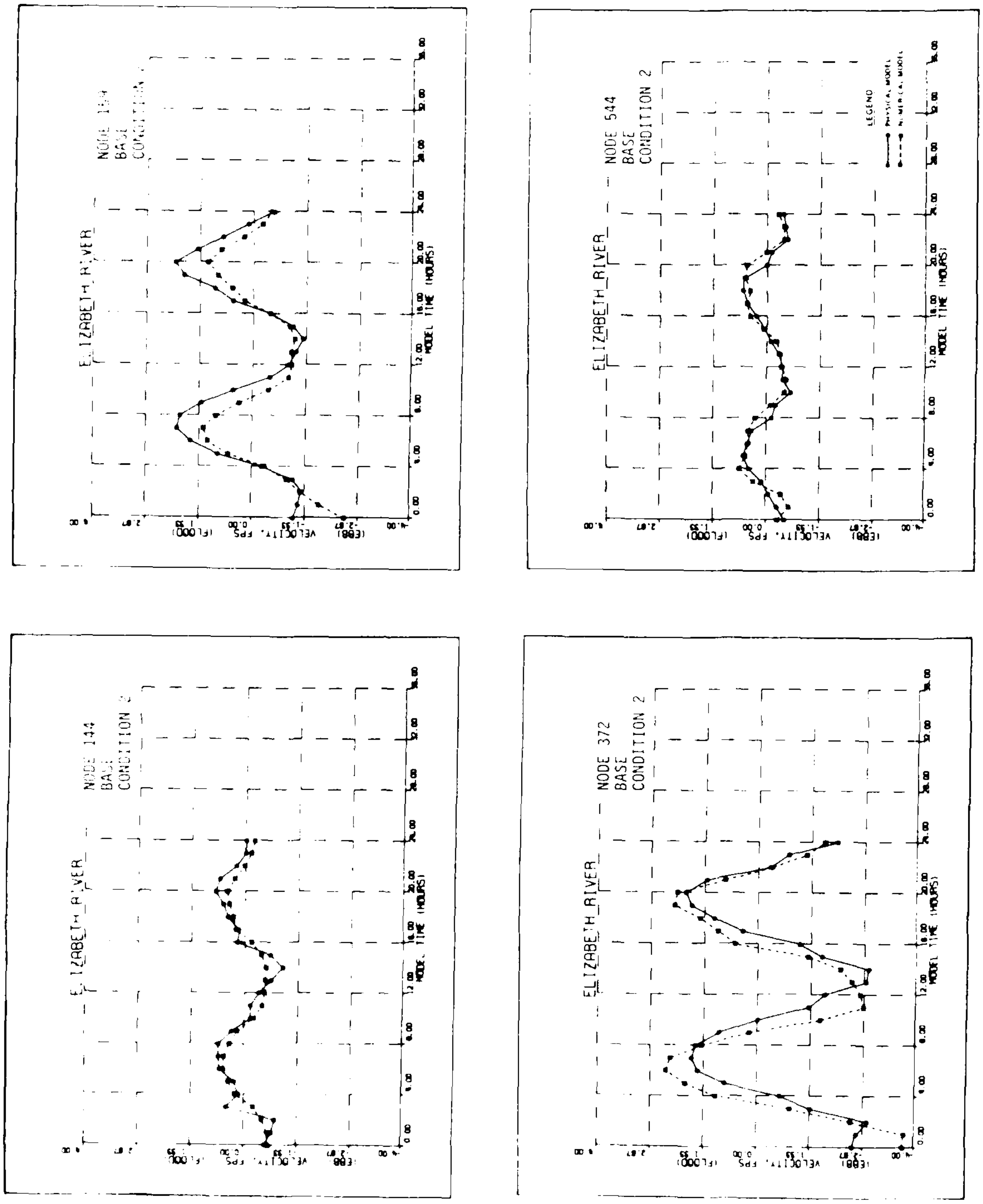
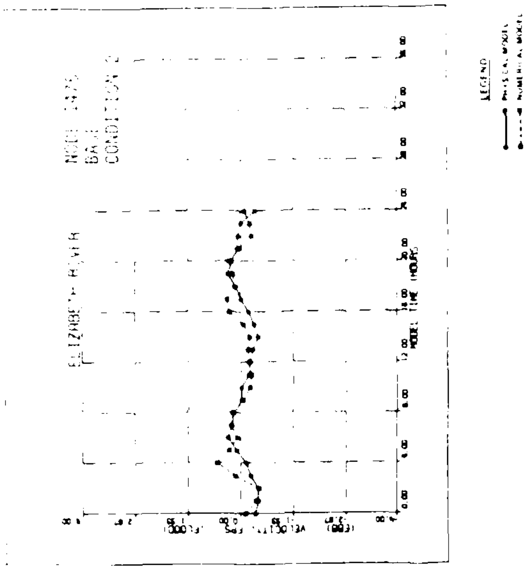

ELIZABETH RIVER
NODE 544
BASE
CONDITION 2
LEGEND
MODEL TIME (HOURS)
VELOCITY, FPS
(FLOOD)
(EBB)
NODE 372
BASE
CONDITION 2

PLATE 5


ELIZABETH RIVER
BASE
CONDITION 2
MODEL TIME (HOURS)
VELOCITY, FPS
(FLOOD)
(EBB)
LEGEND

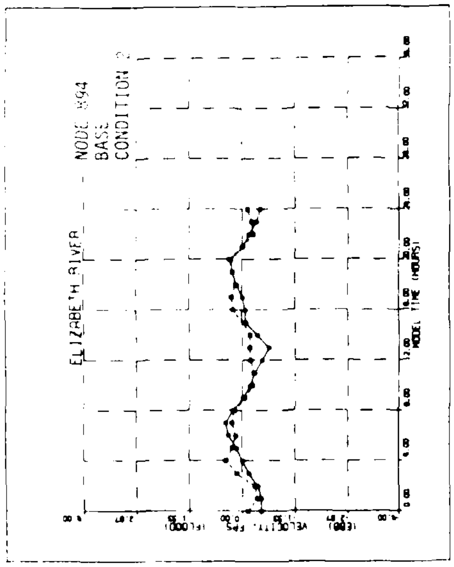

ELIZABETH RIVER
NODE 894
BASE
CONDITION 2
MODEL TIME (HOURS)
VELOCITY, FPS
(FLOOD)
(EBB)

PLATE 6

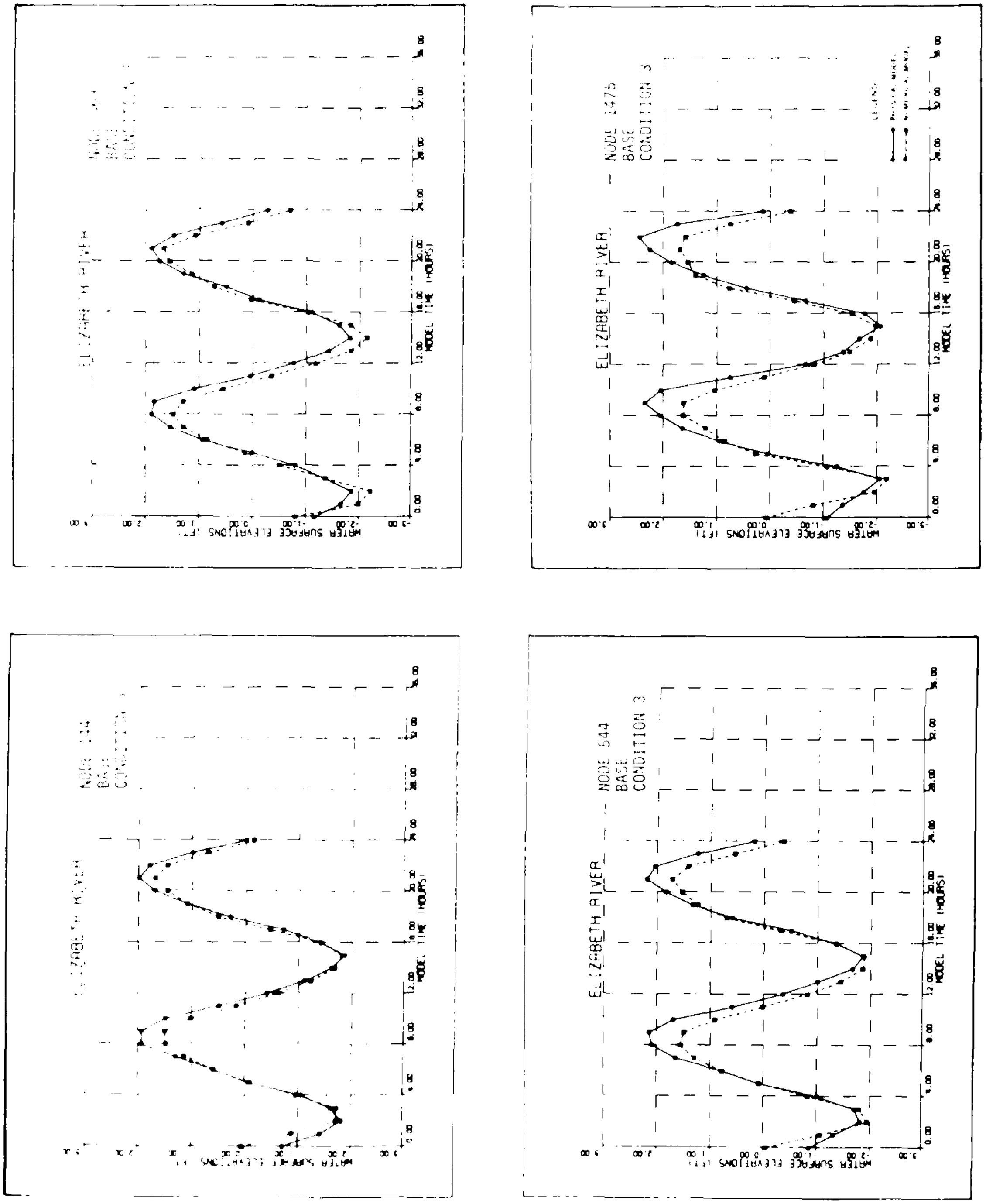

ELIZABETH RIVER
NODE 1475
BASE
CONDITION 3
ELIZABETH RIVER
NODE 544
BASE
CONDITION 3
WATER SURFACE ELEVATIONS (FT)
MODEL TIME (HOURS)

PLATE 7

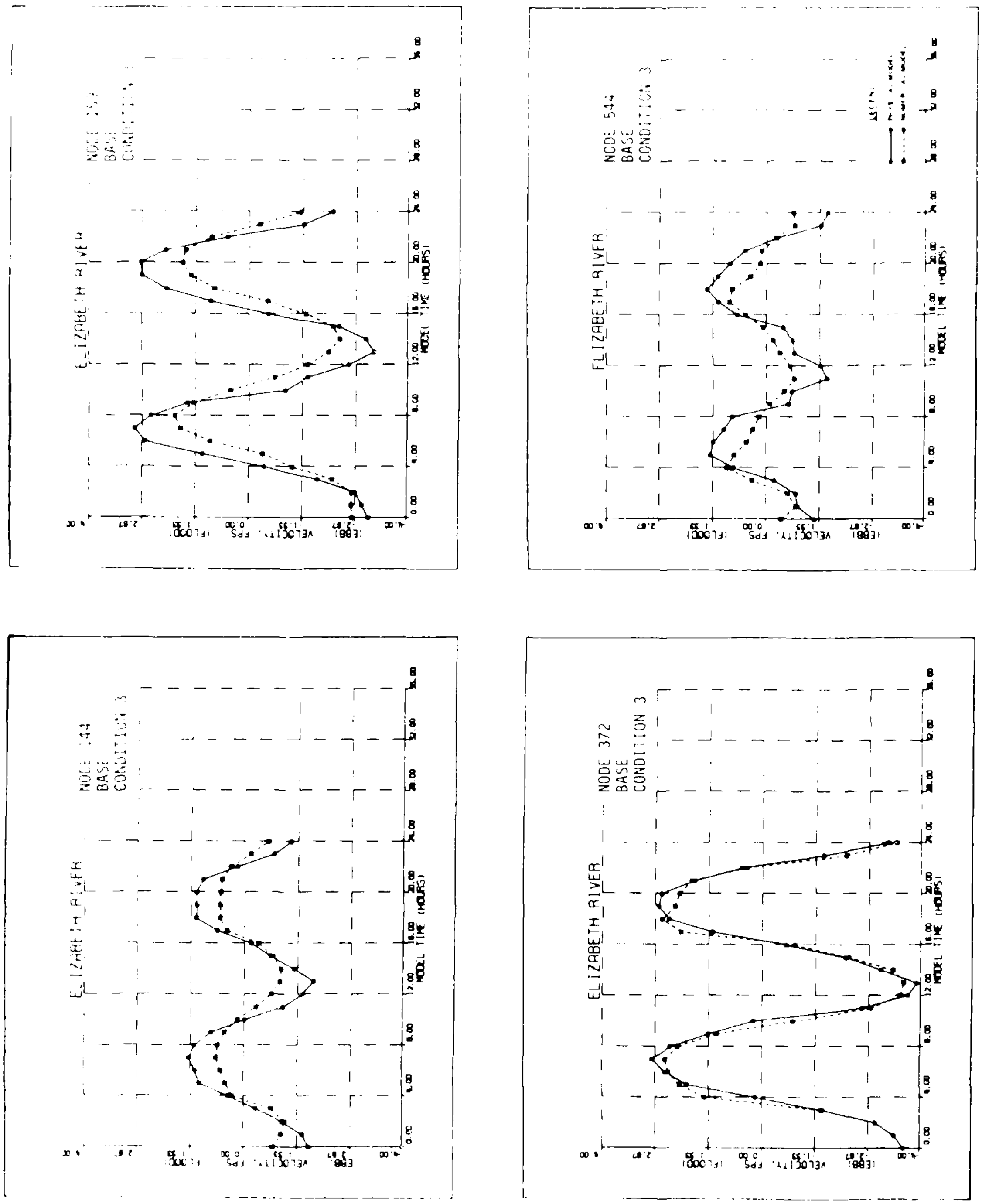
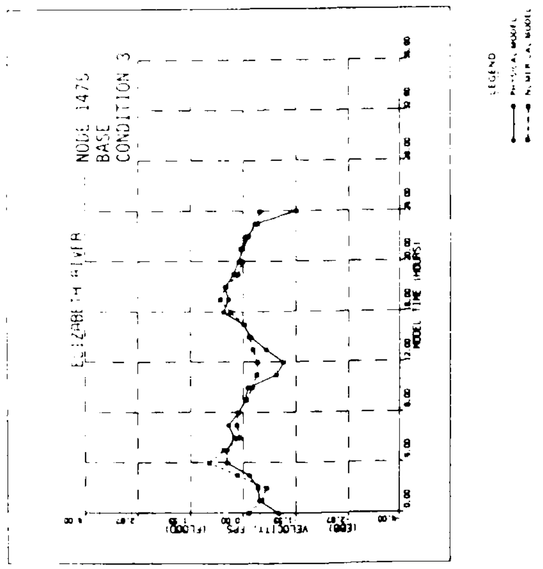

ELIZABETH RIVER
NODE 144
BASE
CONDITION 3
ELIZABETH RIVER
NODE 544
BASE
CONDITION 3
ELIZABETH RIVER
NODE 372
BASE
CONDITION 3
VELOCITY, FPS
MODEL TIME (HOURS)

PLATE 8


ELIZABETH RIVER
NODE 1475
BASE
CONDITION 3
MODEL TIME (HOURS)
VELOCITY, FPS
(FLOOD)
(EBB)
LEGEND

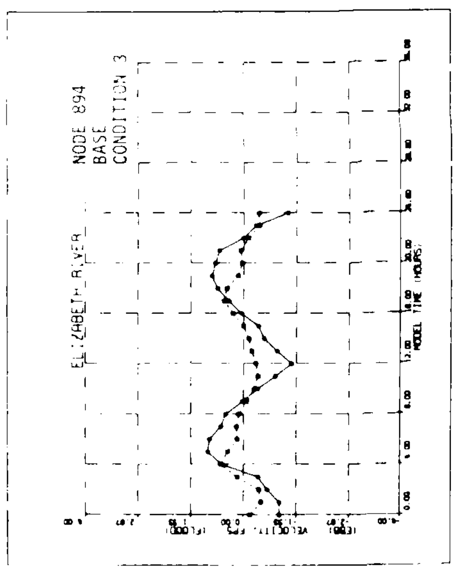

ELIZABETH RIVER
NODE 894
BASE
CONDITION 3
MODEL TIME (HOURS)
VELOCITY, FPS
(FLOOD)
(EBB)

PLATE 9

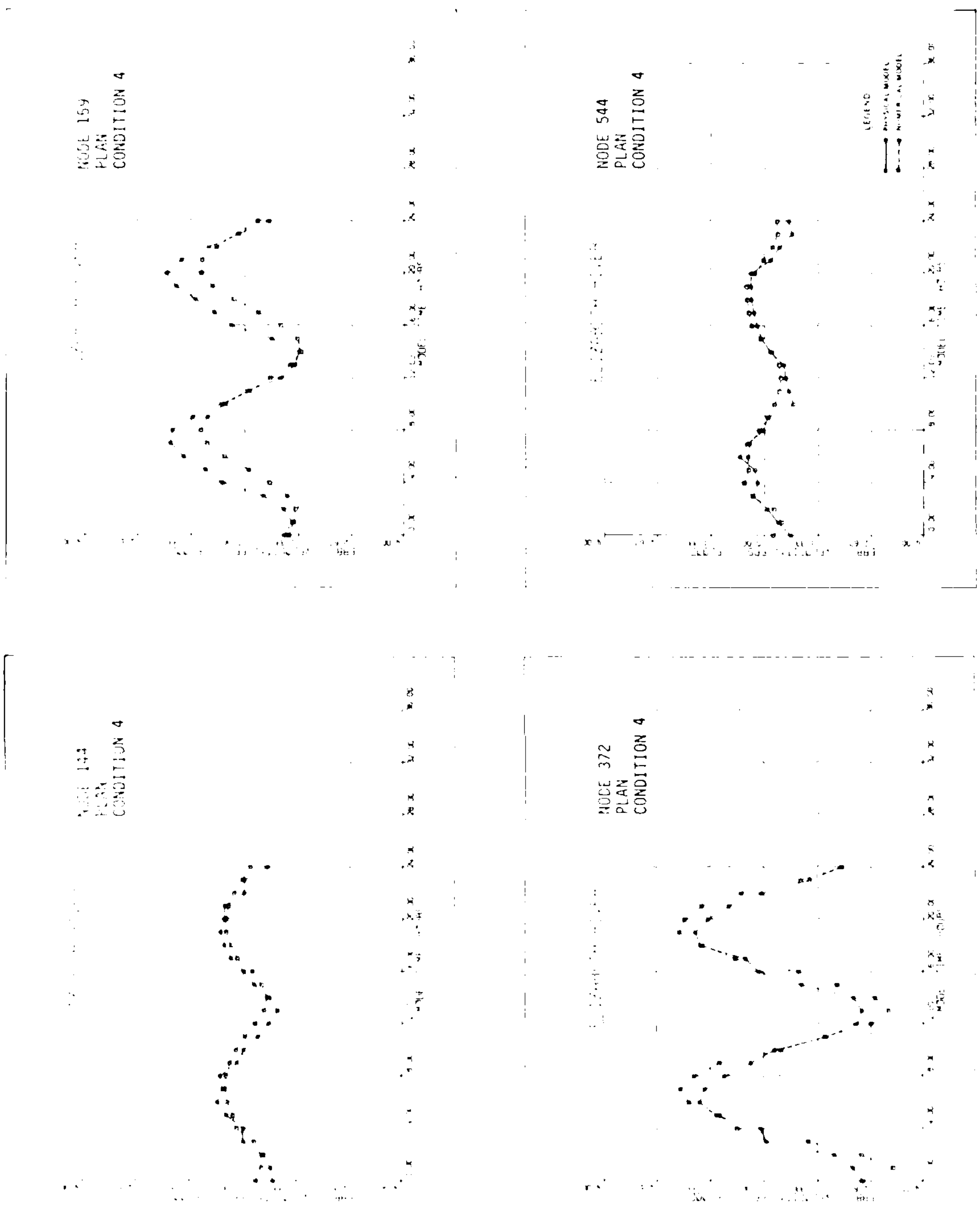

NODE 159
PLAN
CONDITION 4
NODE 544
PLAN
CONDITION 4
CONDITION 4
NODE 372
PLAN
CONDITION 4

PLATE 23

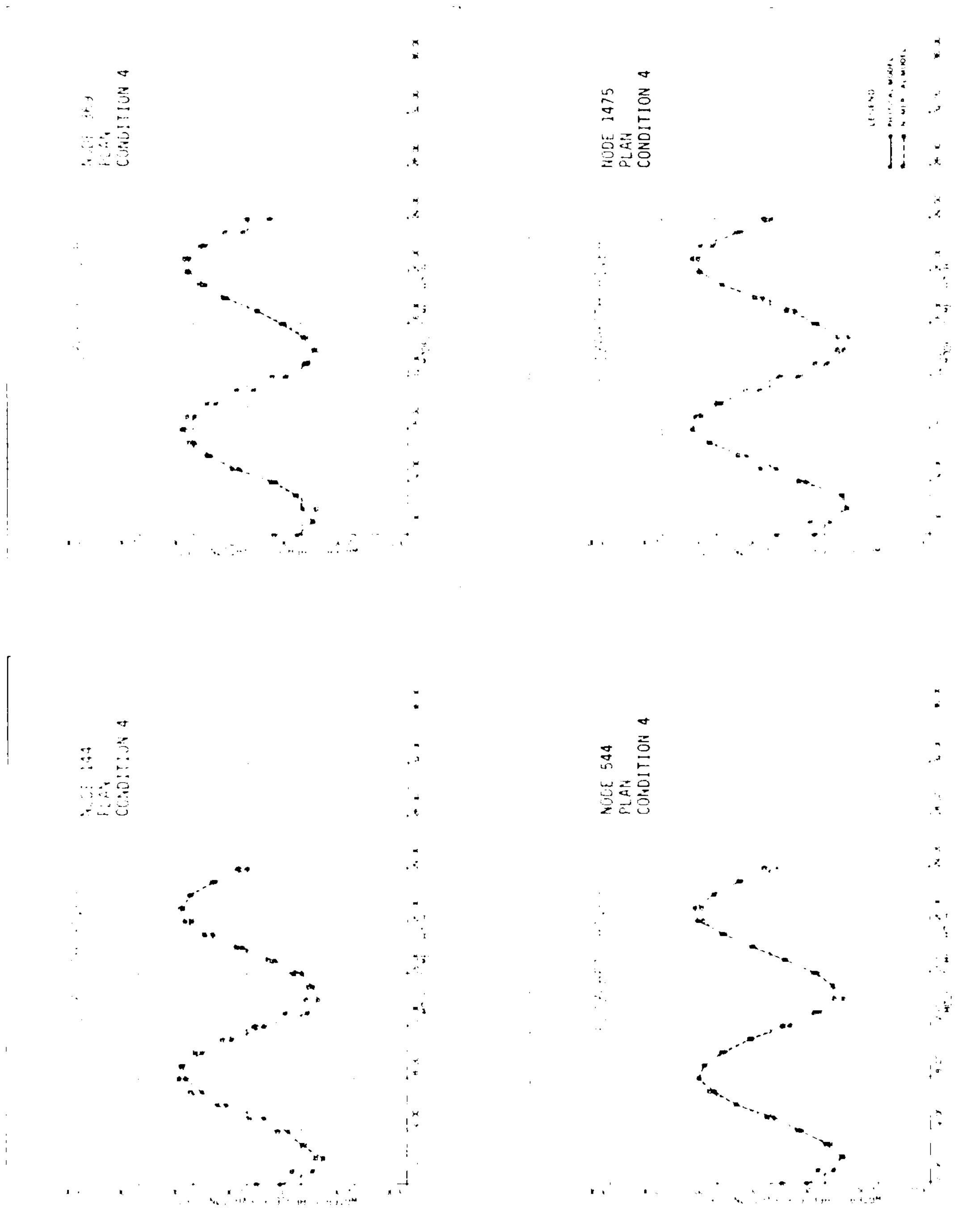
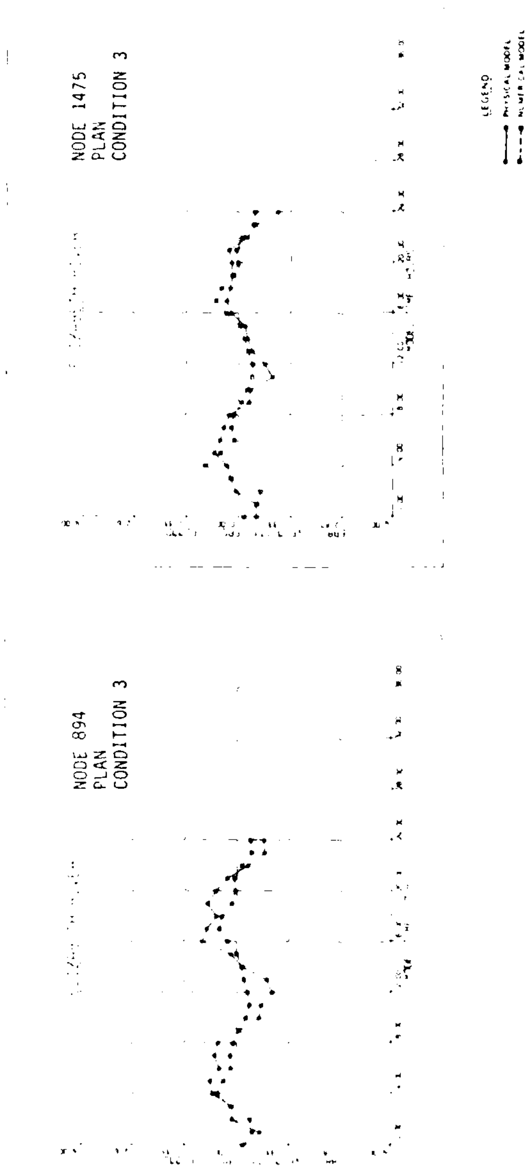


PLATE 22


NODE 1475
PLAN
CONDITION 3
NODE 894
PLAN
CONDITION 3
LEGEND

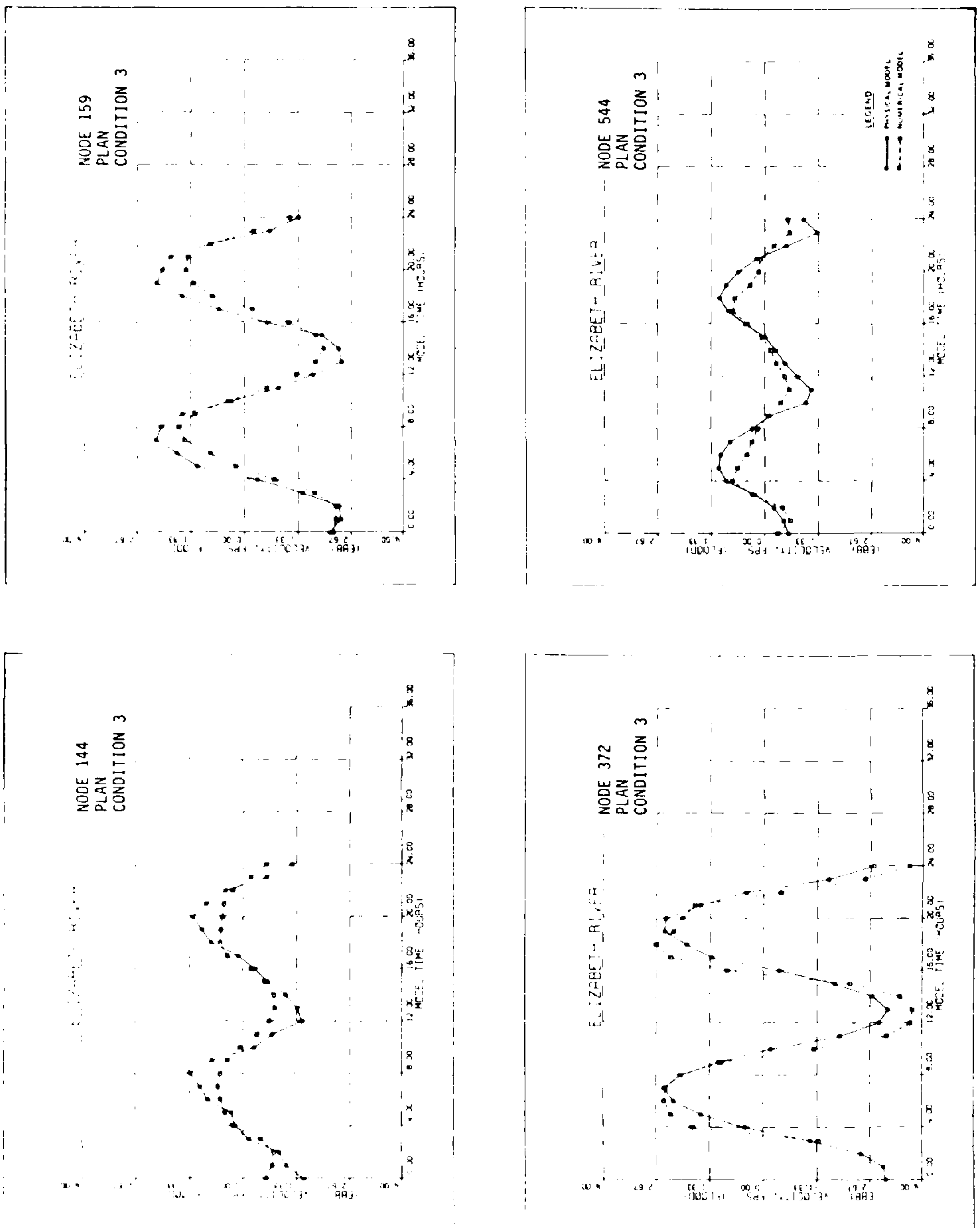

NODE 159
PLAN
CONDITION 3
NODE 144
PLAN
CONDITION 3
NODE 544
PLAN
CONDITION 3
NODE 372
PLAN
CONDITION 3

PLATE 20

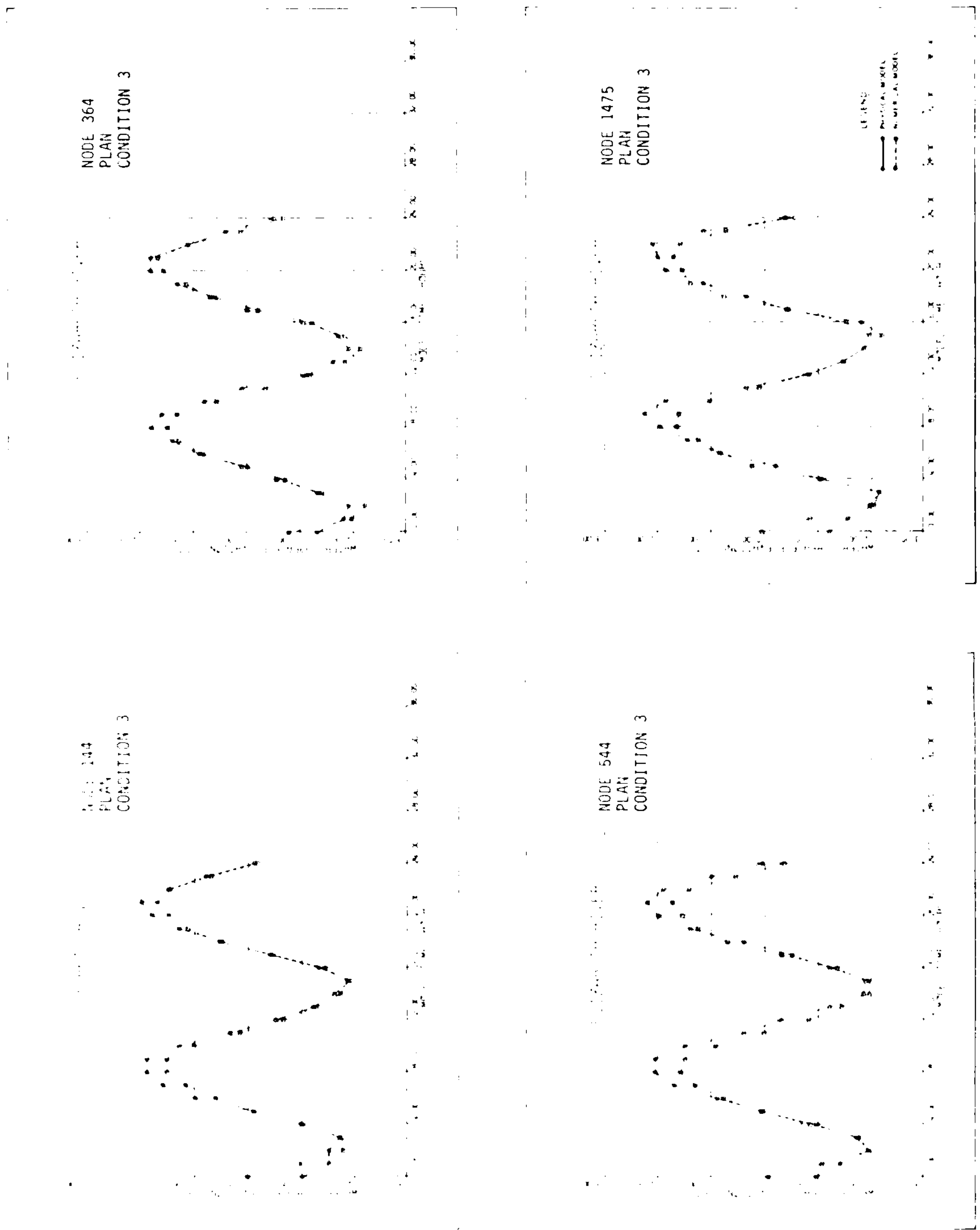

NODE 364
PLAN
CONDITION 3
NODE 1475
PLAN
CONDITION 3
144
PLAN
CONDITION 3
NODE 544
PLAN
CONDITION 3

PLATE 19

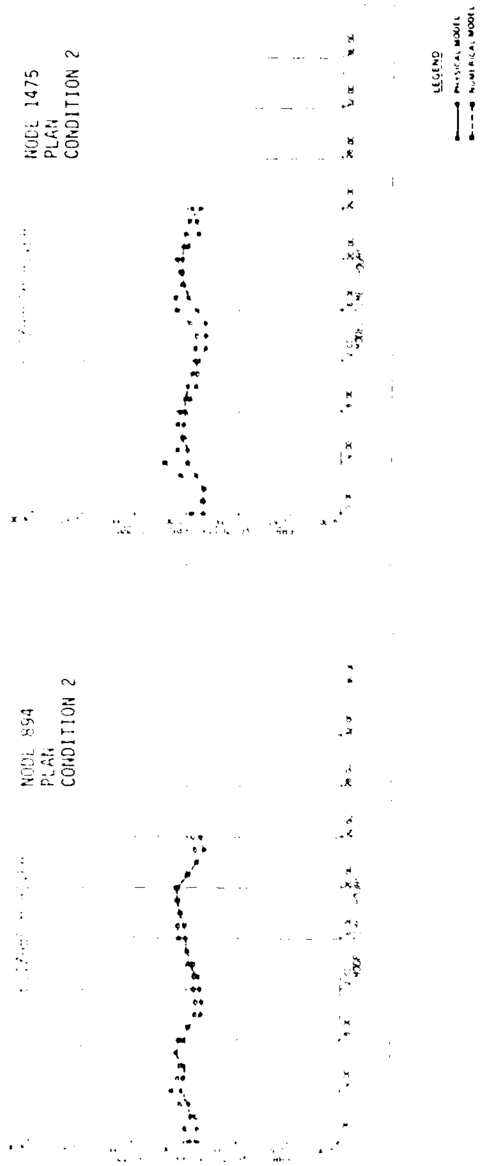
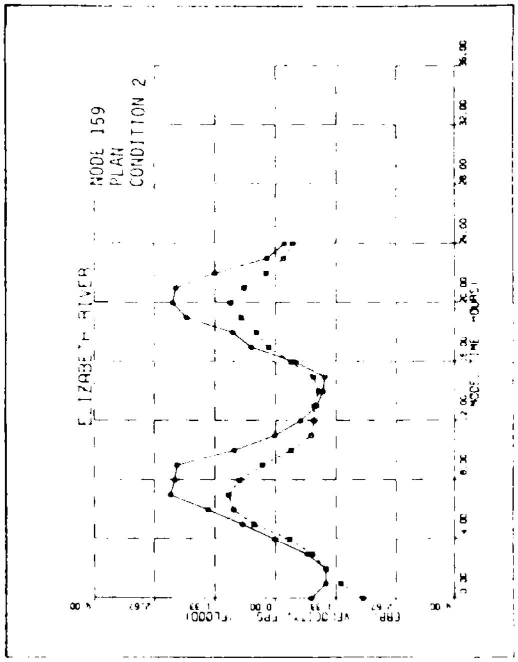
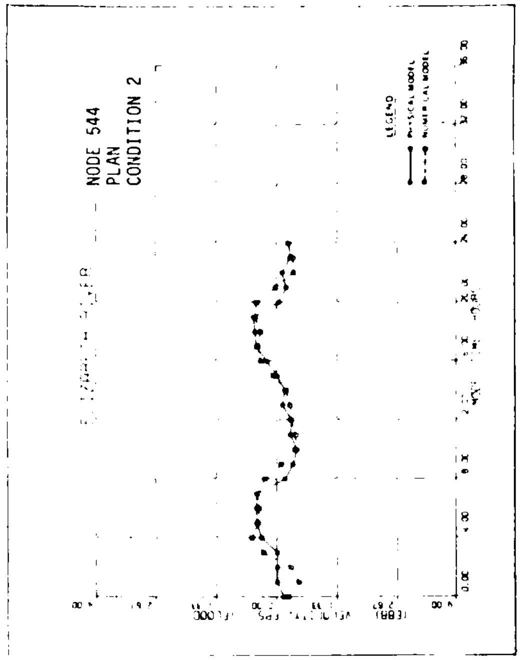
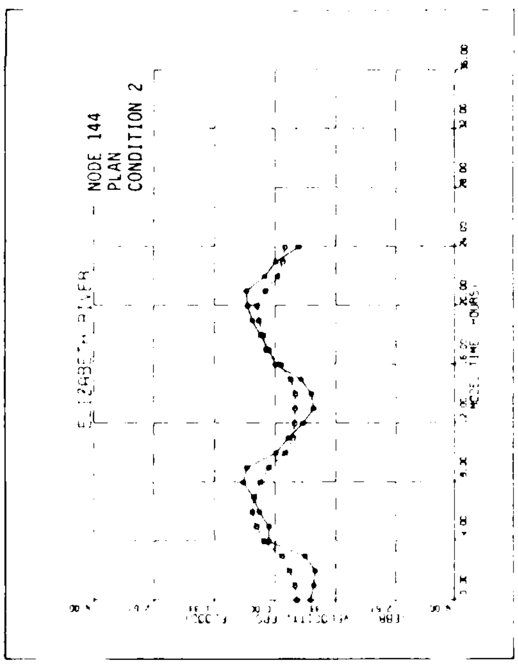

NODE 1475
PLAN
CONDITION 2
NODE 894
PLAN
CONDITION 2
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL

PLATE 18


ELIZABETH RIVER
NODE 159
PLAN
CONDITION 2


ELIZABETH RIVER
NODE 544
PLAN
CONDITION 2
LEGEND


ELIZABETH RIVER
NODE 144
PLAN
CONDITION 2

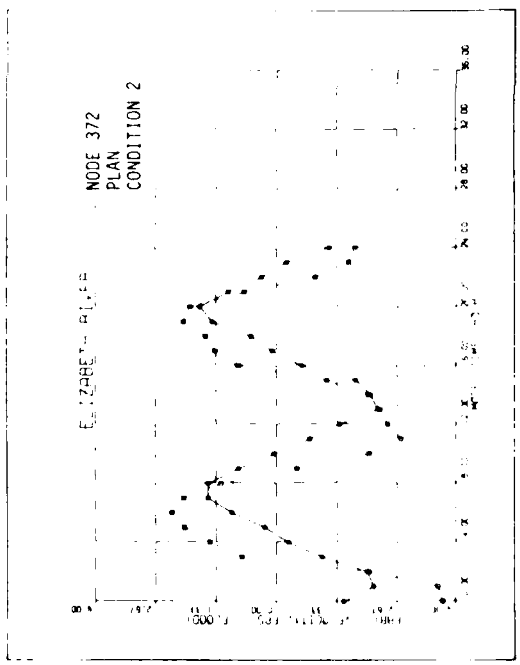
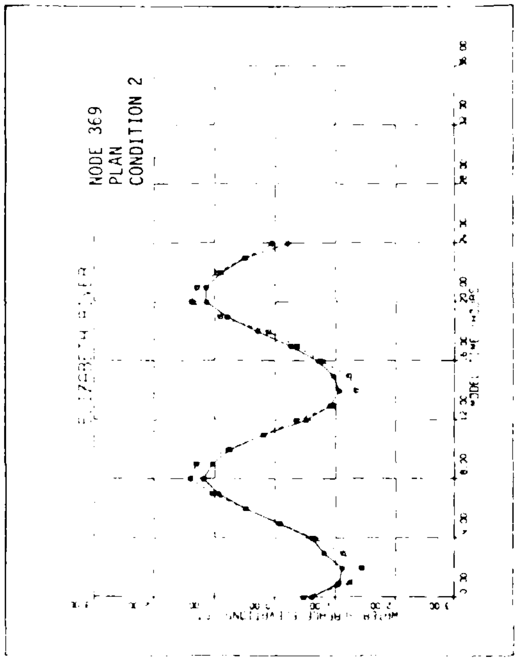
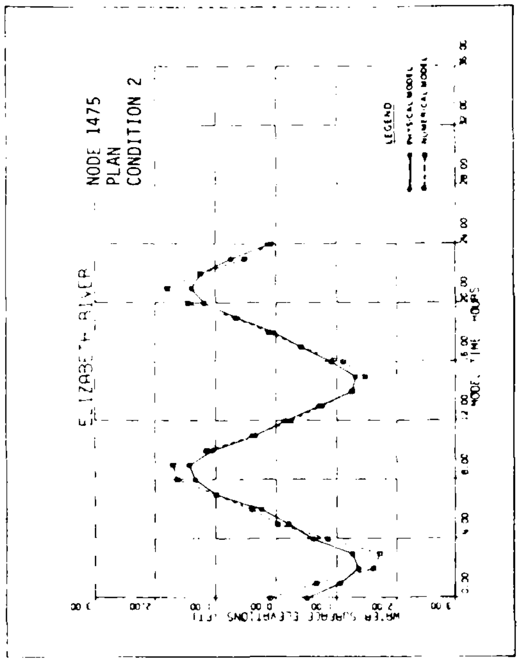
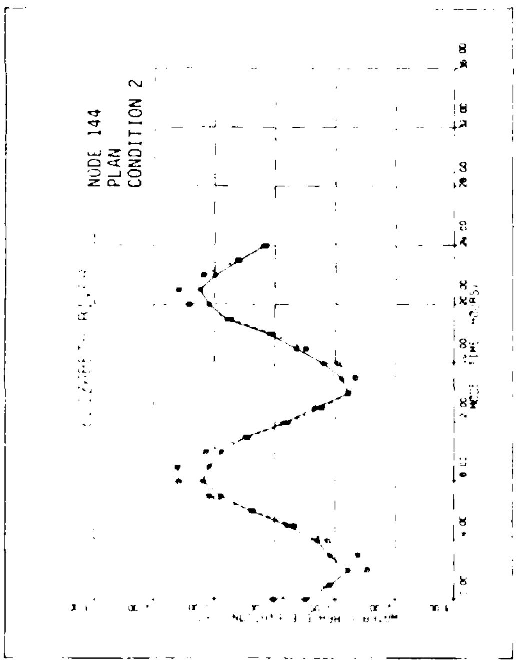

ELIZABETH RIVER
NODE 372
PLAN
CONDITION 2

PLATE 17


NODE 369
PLAN
CONDITION 2


NODE 1475
PLAN
CONDITION 2
LEGEND


NODE 144
PLAN
CONDITION 2

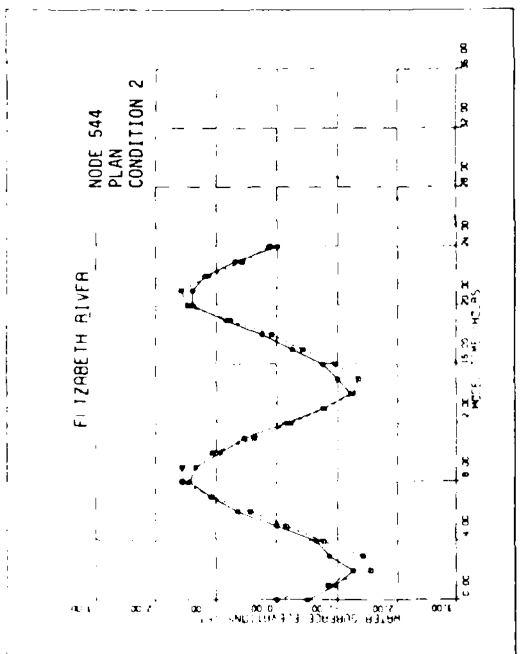
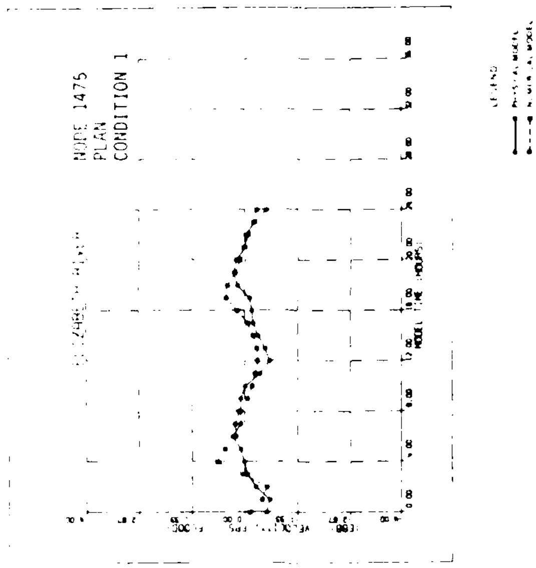

NODE 544
PLAN
CONDITION 2

PLATE 16


ELIZABETH RIVER
NODE 1475
PLAN
CONDITION 1
MODEL TIME (HOURS)
VELOCITY, FPS

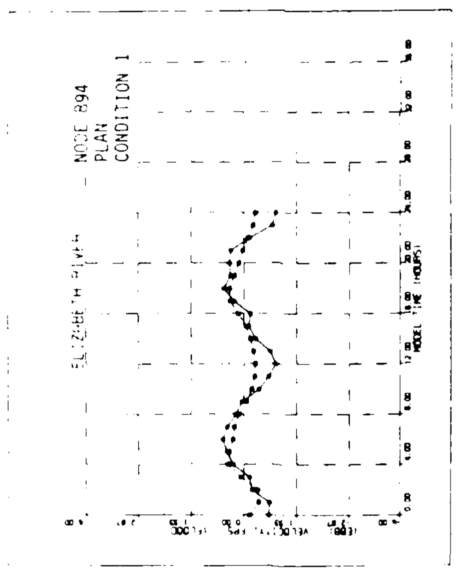
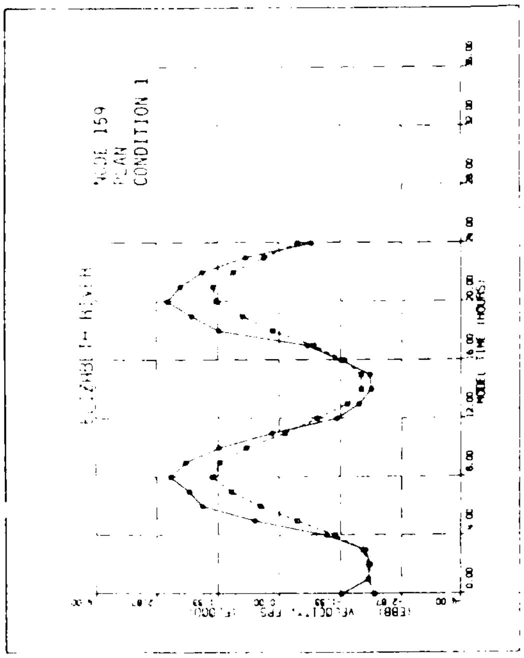
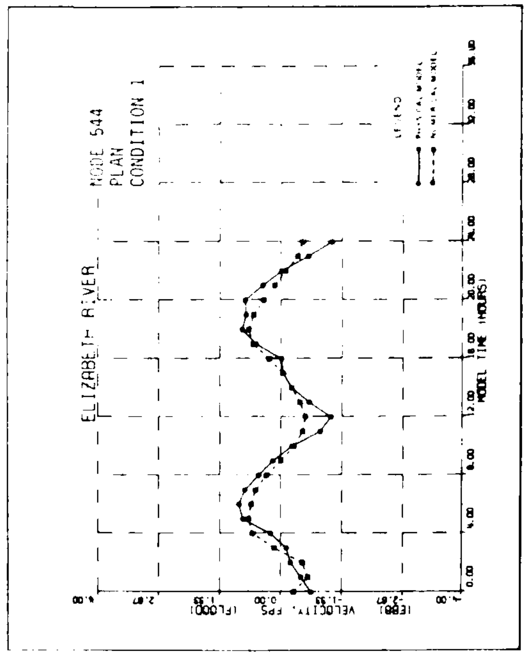
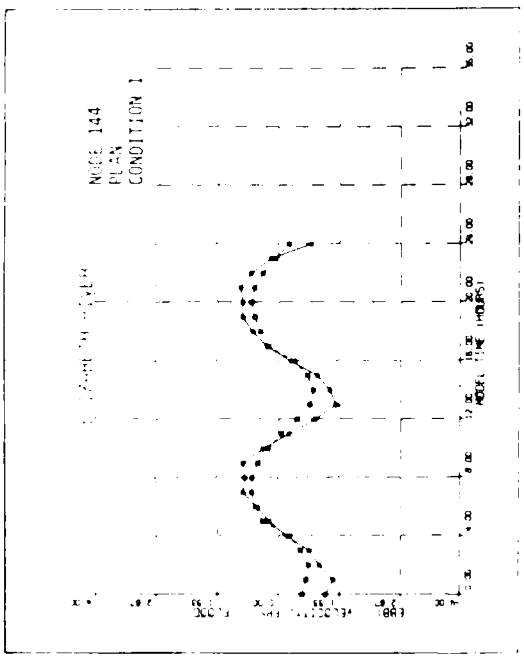

ELIZABETH RIVER
NODE 894
PLAN
CONDITION 1
MODEL TIME (HOURS)
VELOCITY, FPS

PLATE 15


ELIZABETH RIVER
NODE 159
PLAN
CONDITION 1
(EBB) VELOCITY, FPS (FLOOD)
MODEL TIME (HOURS)


ELIZABETH RIVER
NODE 544
PLAN
CONDITION 1
(EBB) VELOCITY, FPS (FLOOD)
MODEL TIME (HOURS)


NODE 144
PLAN
CONDITION 1
MODEL TIME (HOURS)

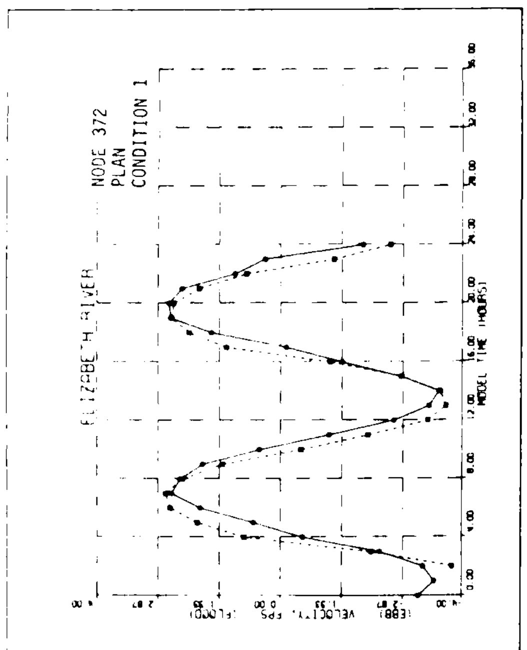
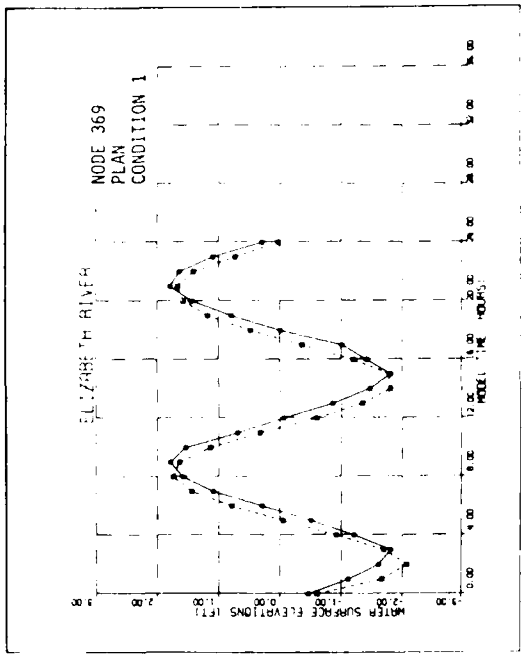
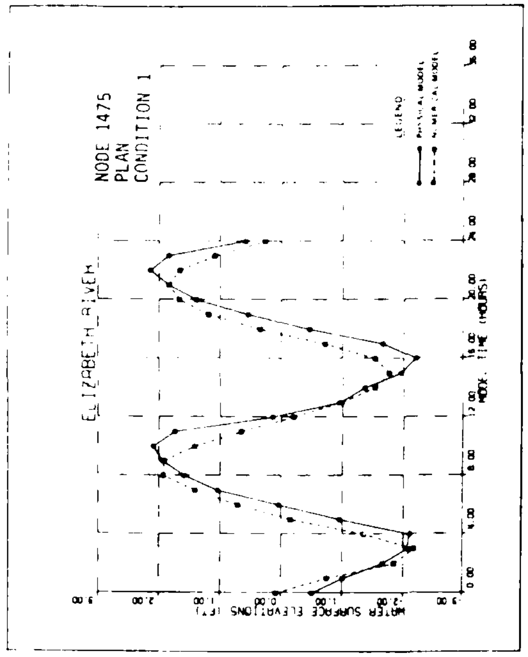
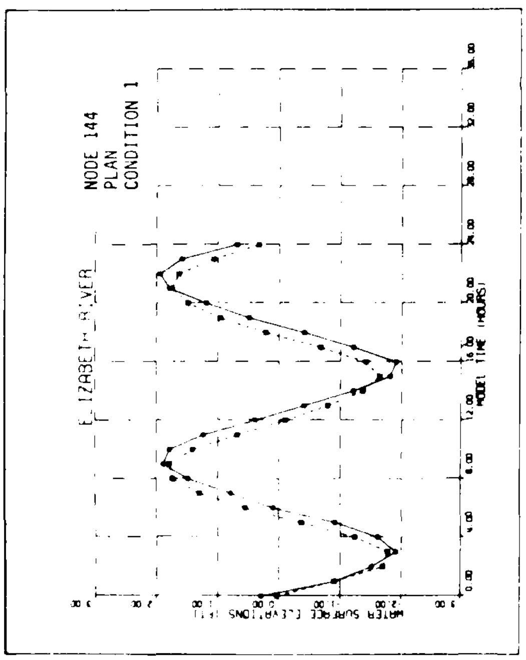

ELIZABETH RIVER
NODE 372
PLAN
CONDITION 1
(EBB) VELOCITY, FPS (FLOOD)
MODEL TIME (HOURS)

PLATE 14


ELIZABETH RIVER
NODE 369
PLAN
CONDITION 1
WATER SURFACE ELEVATIONS (FT)
MODEL TIME (HOURS)


ELIZABETH RIVER
NODE 1475
PLAN
CONDITION 1
WATER SURFACE ELEVATIONS (FT)
MODEL TIME (HOURS)


ELIZABETH RIVER
NODE 144
PLAN
CONDITION 1
WATER SURFACE ELEVATIONS (FT)
MODEL TIME (HOURS)

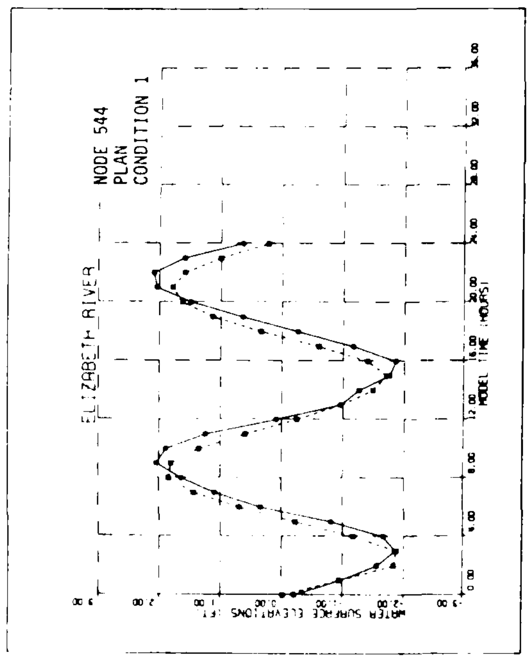
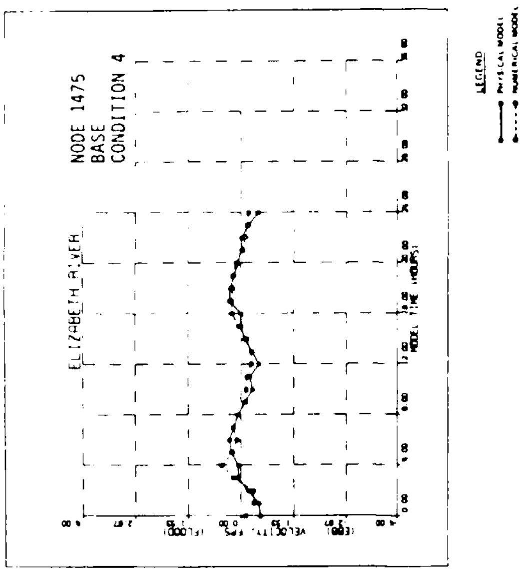

ELIZABETH RIVER
NODE 544
PLAN
CONDITION 1
WATER SURFACE ELEVATIONS (FT)
MODEL TIME (HOURS)

PLATE 13


ELIZABETH RIVER
NODE 1475
BASE
CONDITION 4
VELOCITY, FPS
MODEL TIME (HOURS)
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL

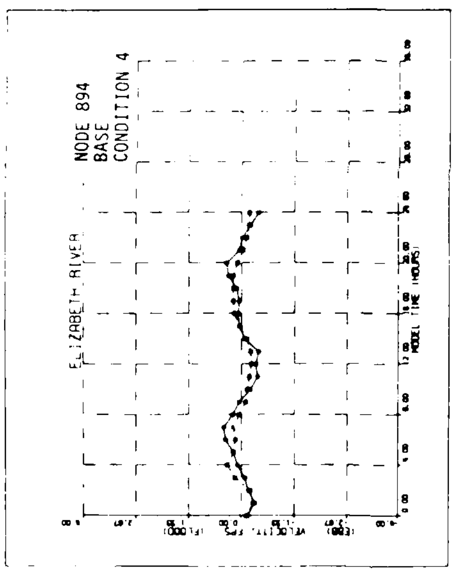

ELIZABETH RIVER
NODE 894
BASE
CONDITION 4
VELOCITY, FPS
MODEL TIME (HOURS)

PLATE 12

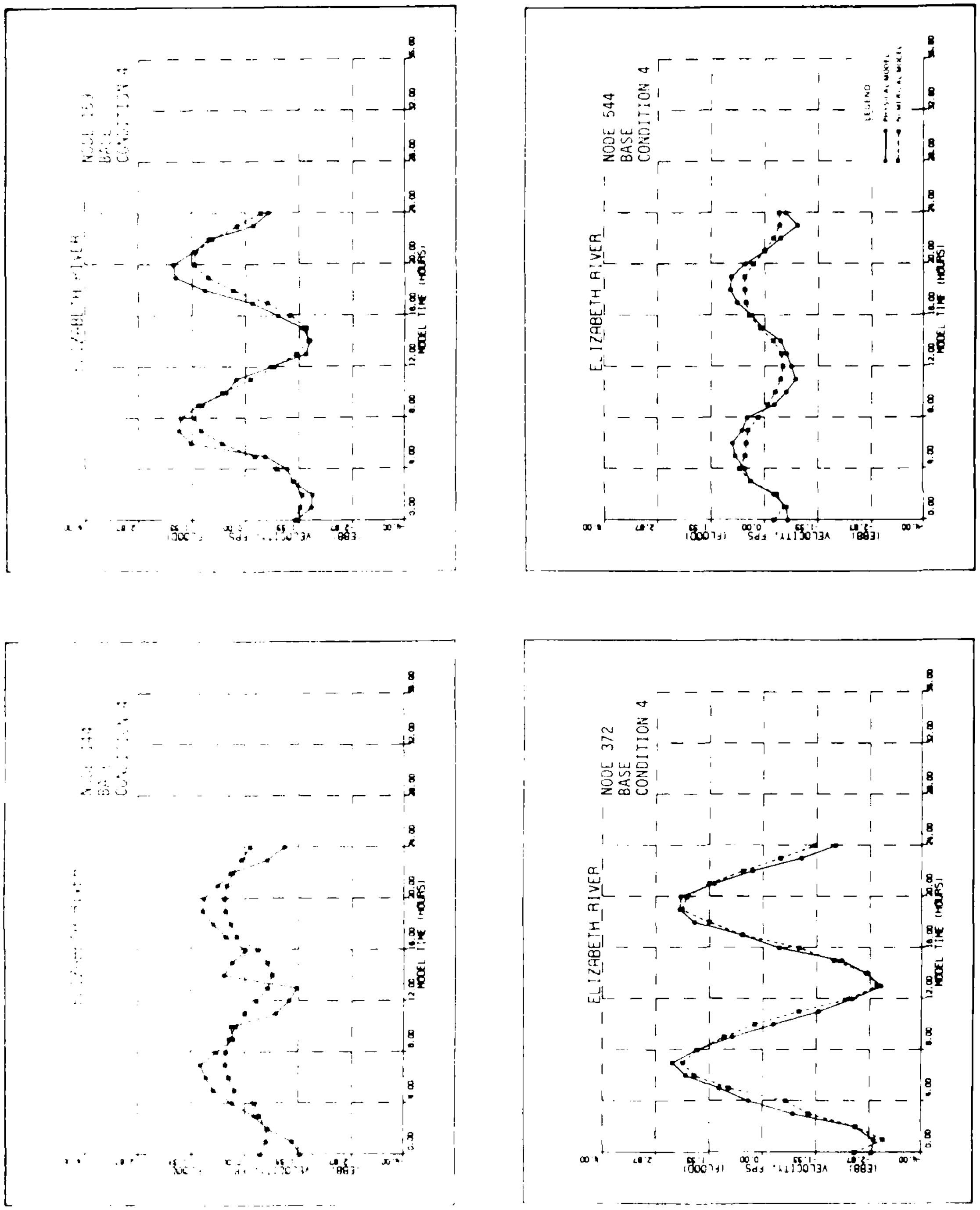

ELIZABETH RIVER
NODE 544
BASE
CONDITION 4
ELIZABETH RIVER
NODE 372
BASE
CONDITION 4
MODEL TIME (HOURS)
VELOCITY, FPS
(FLOOD)
(EBB)

PLATE 11

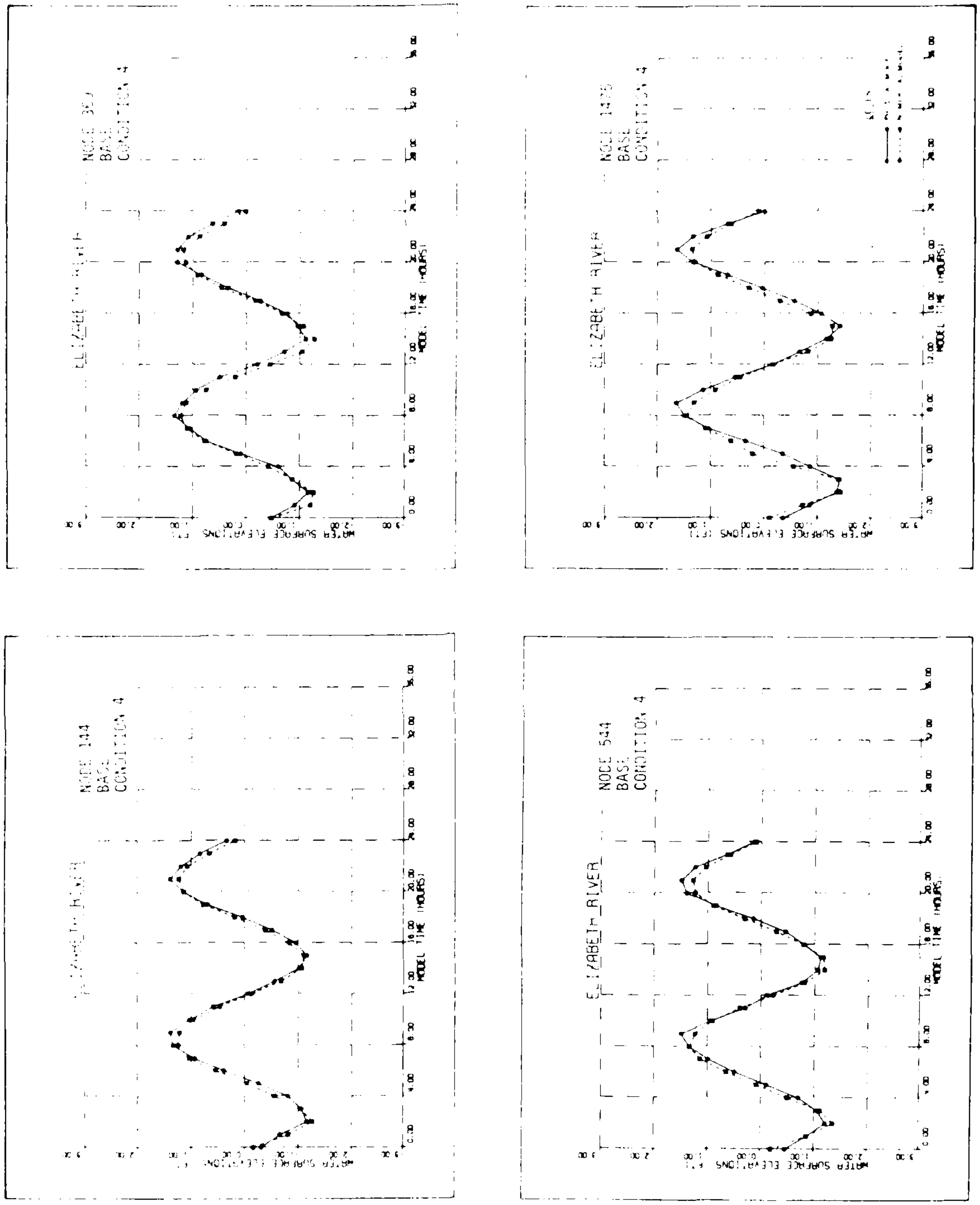

ELIZABETH RIVER
NODE 544
BASE
CONDITION 4
BASE
CONDITION 4
MODEL TIME (HOURS)
WATER SURFACE ELEVATIONS (FT)

PLATE 10

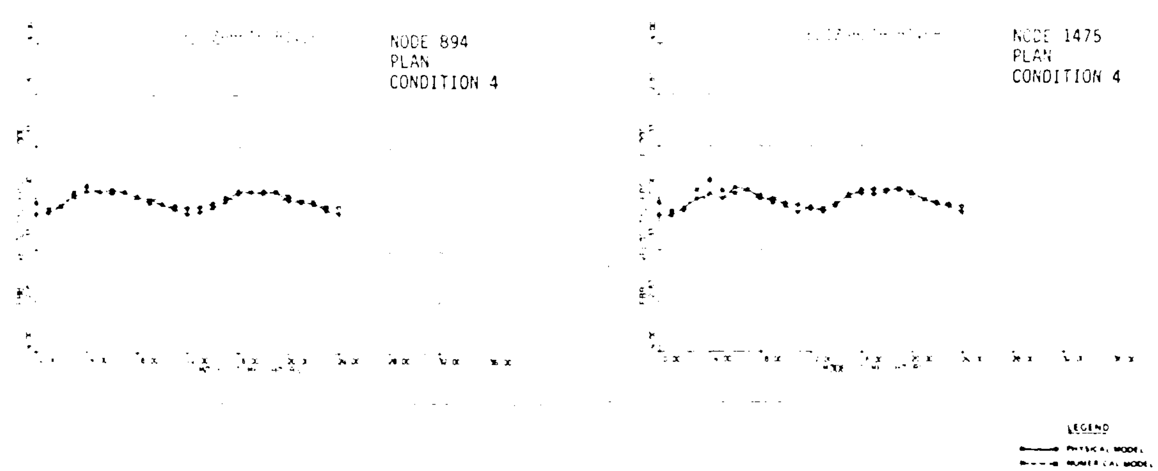
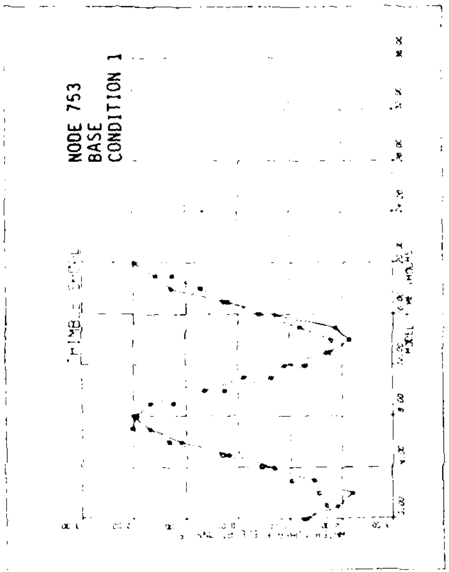
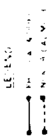
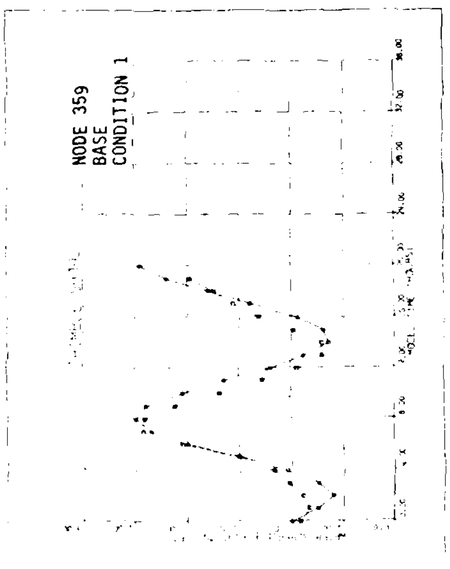

NODE 894
PLAN
CONDITION 4
NODE 1475
PLAN
CONDITION 4
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL

PLATE 24


NODE 753
BASE
CONDITION 1




NODE 359
BASE
CONDITION 1

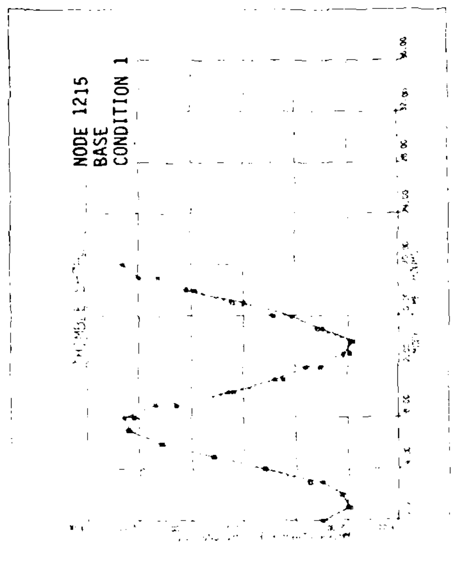

NODE 1215
BASE
CONDITION 1

PLATE 25

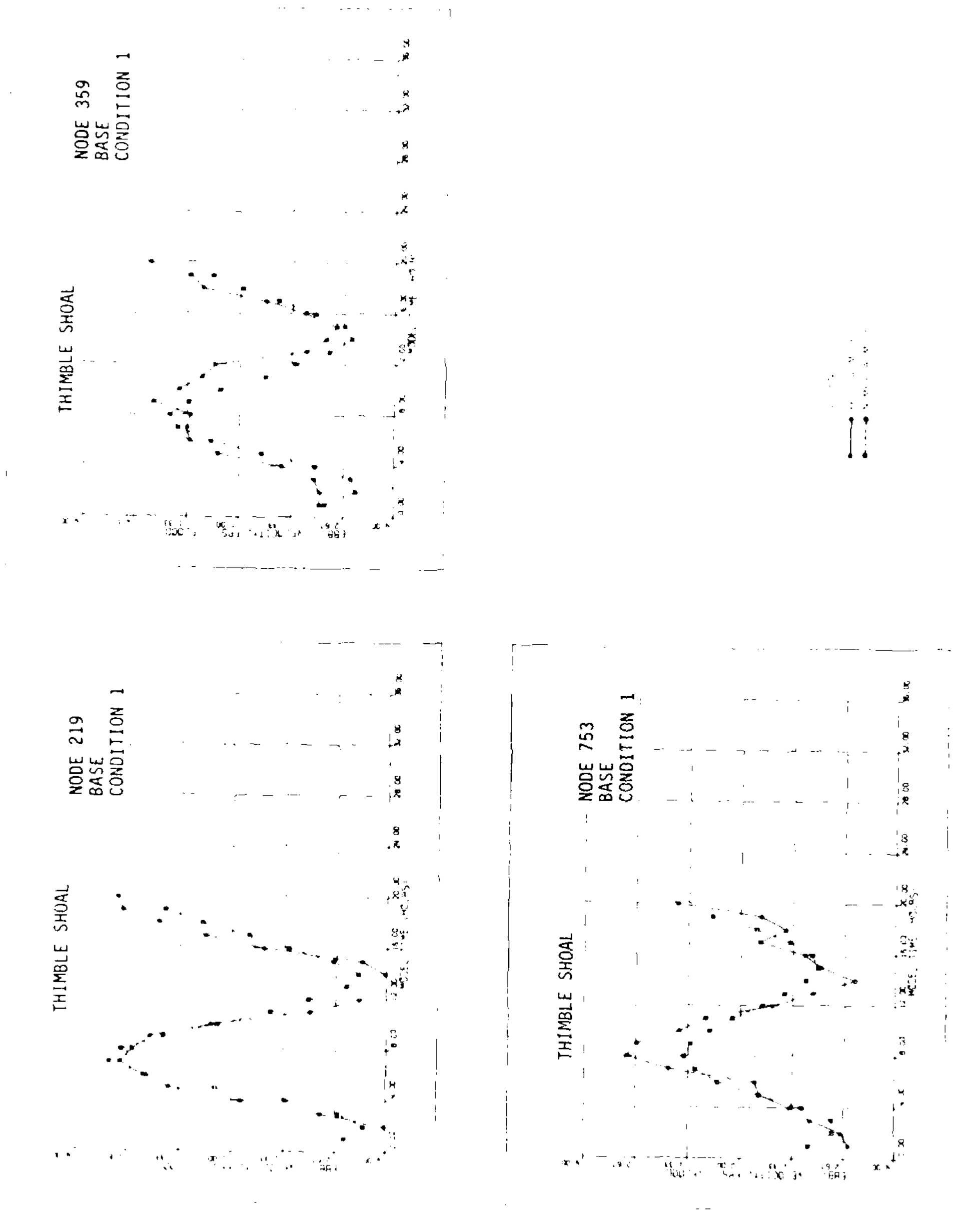
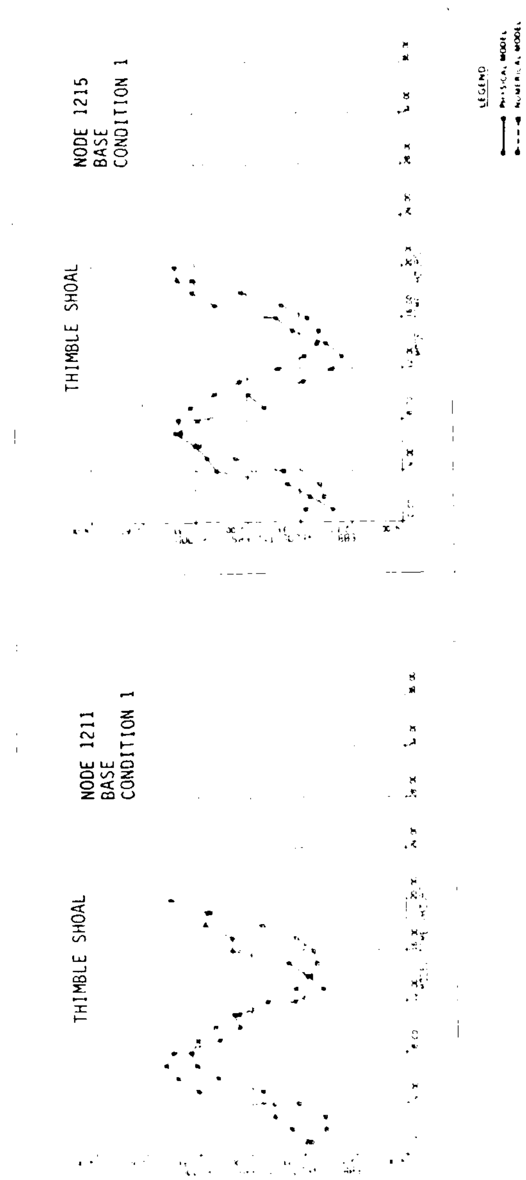
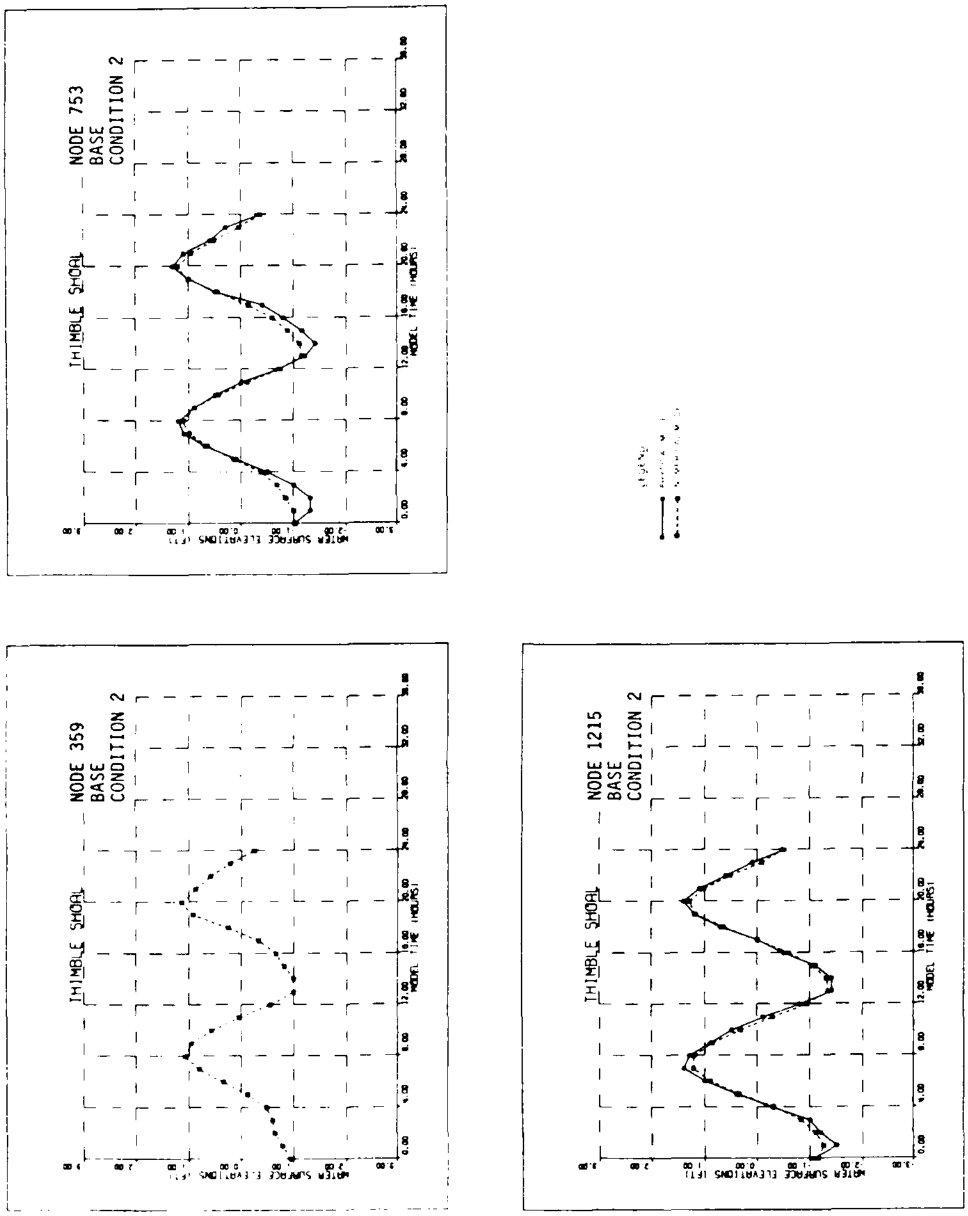

THIMBLE SHOAL
NODE 359
BASE
CONDITION 1
THIMBLE SHOAL
NODE 219
BASE
CONDITION 1
THIMBLE SHOAL
NODE 753
BASE
CONDITION 1

PLATE 26


THIMBLE SHOAL
NODE 1215
BASE
CONDITION 1
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL
THIMBLE SHOAL
NODE 1211
BASE
CONDITION 1



THIMBLE SHOAL
NODE 753
BASE
CONDITION 2
THIMBLE SHOAL
NODE 359
BASE
CONDITION 2
THIMBLE SHOAL
NODE 1215
BASE
CONDITION 2
WATER SURFACE ELEVATIONS (FT)
MODEL TIME (HOURS)

PLATE 28

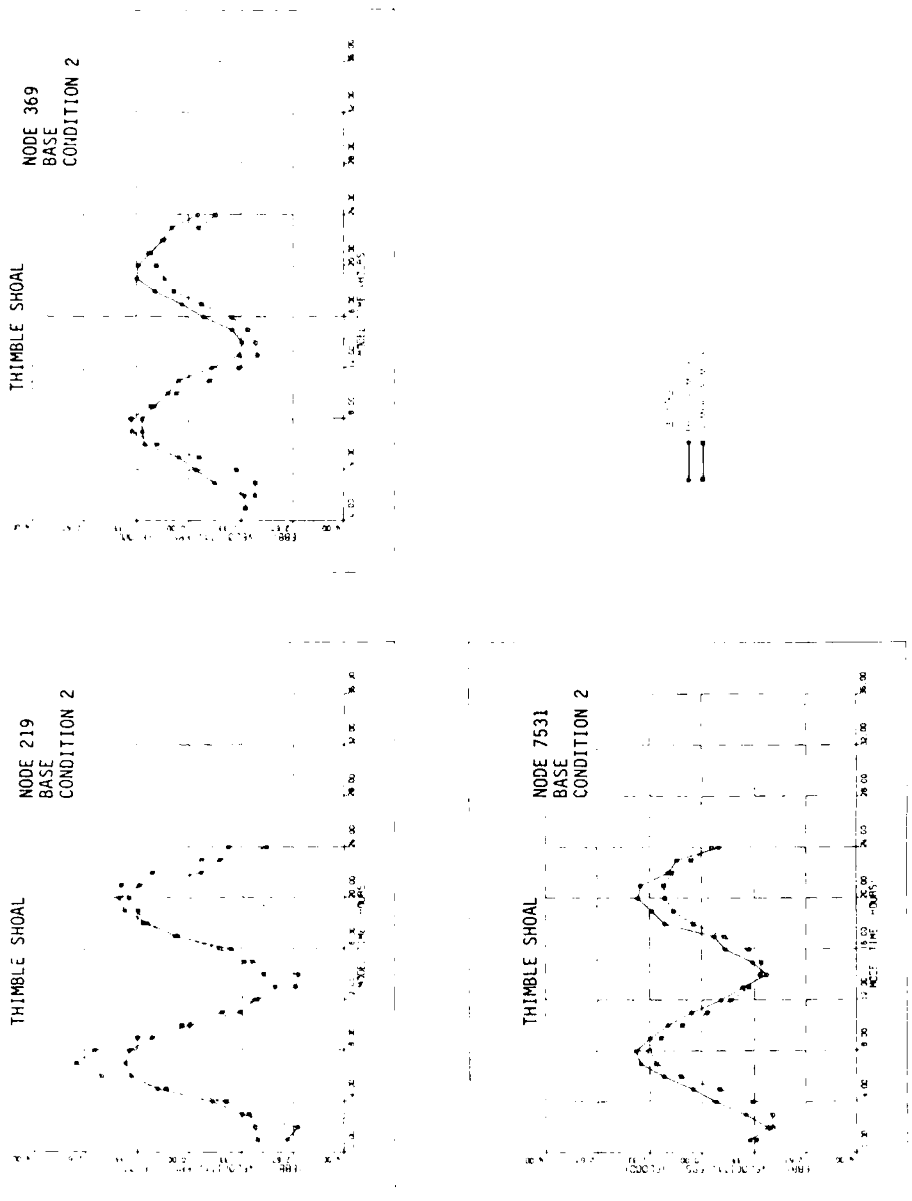
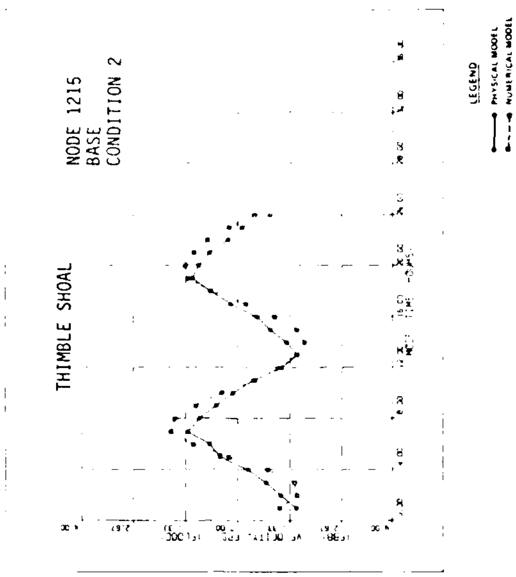

THIMBLE SHOAL
NODE 369
BASE
CONDITION 2
THIMBLE SHOAL
NODE 219
BASE
CONDITION 2
THIMBLE SHOAL
NODE 7531
BASE
CONDITION 2

PLATE 29


THIMBLE SHOAL
NODE 1215
BASE
CONDITION 2
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL

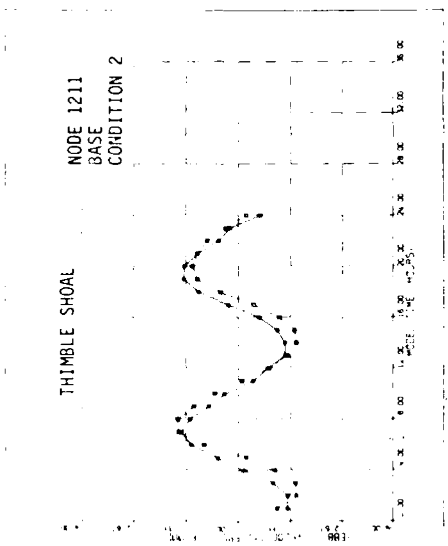
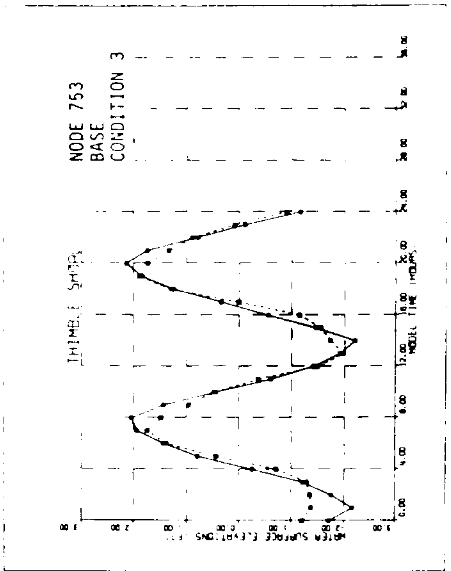
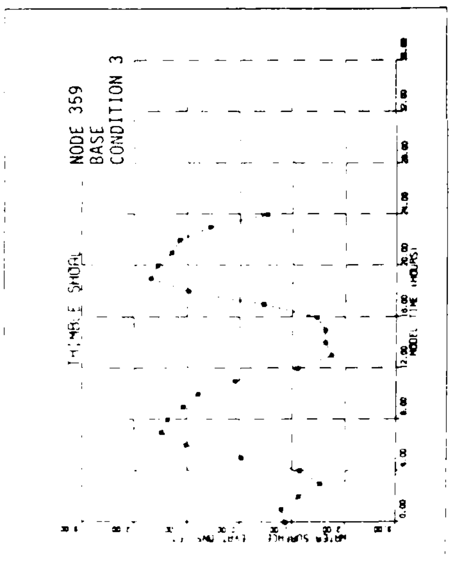
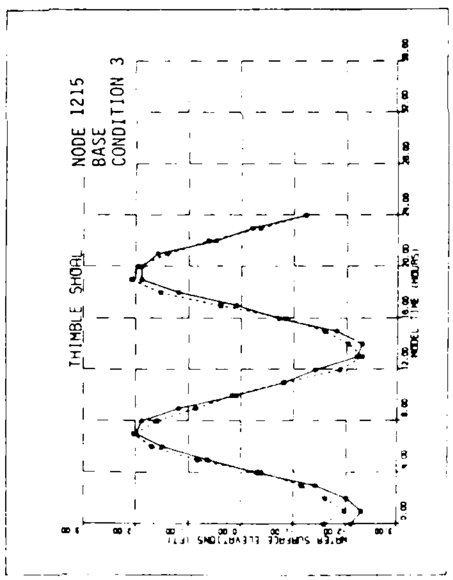
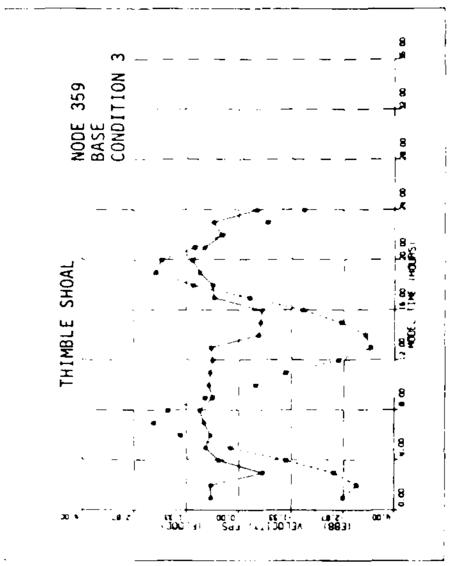

THIMBLE SHOAL
NODE 1211
BASE
CONDITION 2

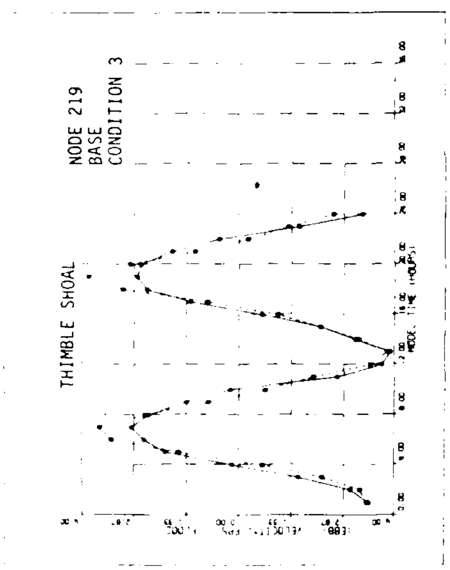
PLATE 30

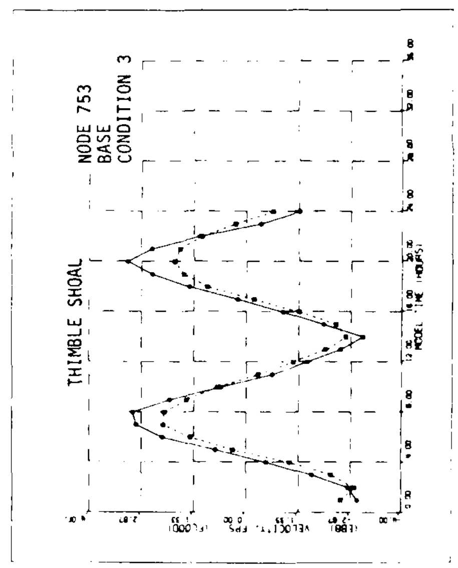

THIMBLE SHOAL
NODE 753
BASE
CONDITION 3
MODEL TIME (HOURS)
WATER SURFACE ELEVATIONS (FT)

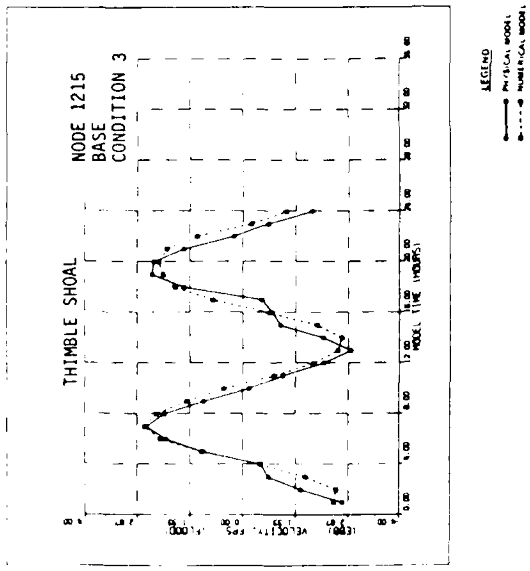

THIMBLE SHOAL
NODE 359
BASE
CONDITION 3
MODEL TIME (HOURS)

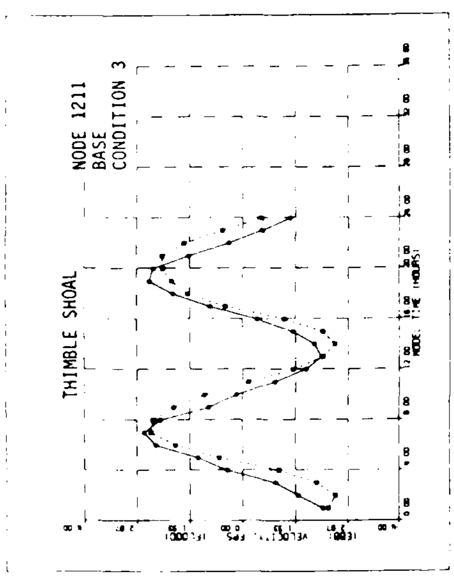

THIMBLE SHOAL
NODE 1215
BASE
CONDITION 3
MODEL TIME (HOURS)
WATER SURFACE ELEVATIONS (FT)

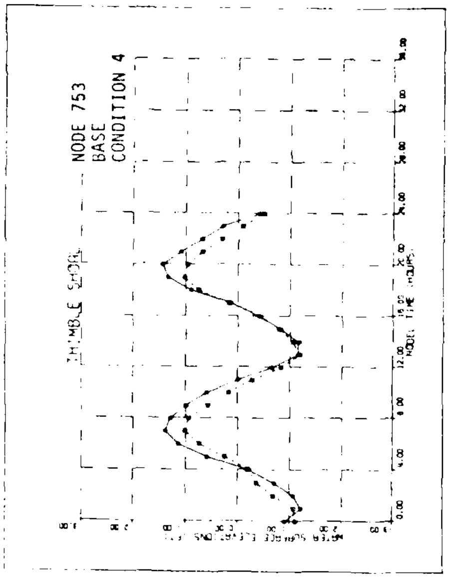

THIMBLE SHOAL
NODE 359
BASE
CONDITION 3

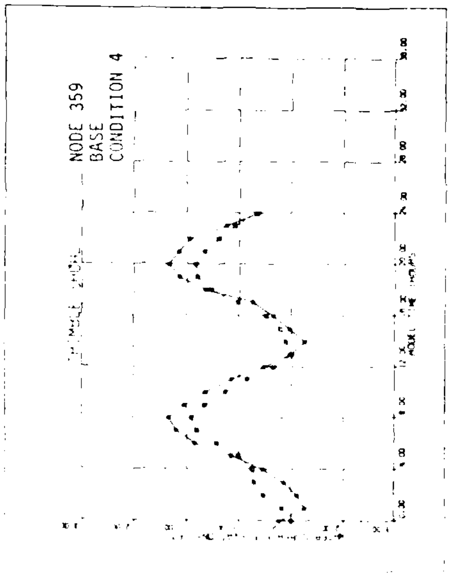

THIMBLE SHOAL
NODE 219
BASE
CONDITION 3


THIMBLE SHOAL
NODE 753
BASE
CONDITION 3


THIMBLE SHOAL
NODE 1215
BASE
CONDITION 3
LEGEND


THIMBLE SHOAL
NODE 1211
BASE
CONDITION 3


THIMBLE SHOAL
NODE 753
BASE
CONDITION 4
MODEL TIME (HOURS)
WATER SURFACE ELEVATIONS, FT


NODE 359
BASE
CONDITION 4
MODEL TIME (HOURS)

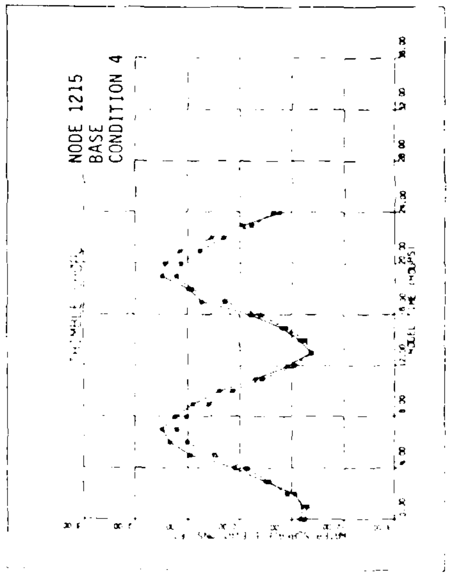

NODE 1215
BASE
CONDITION 4
MODEL TIME (HOURS)

PLATE 34

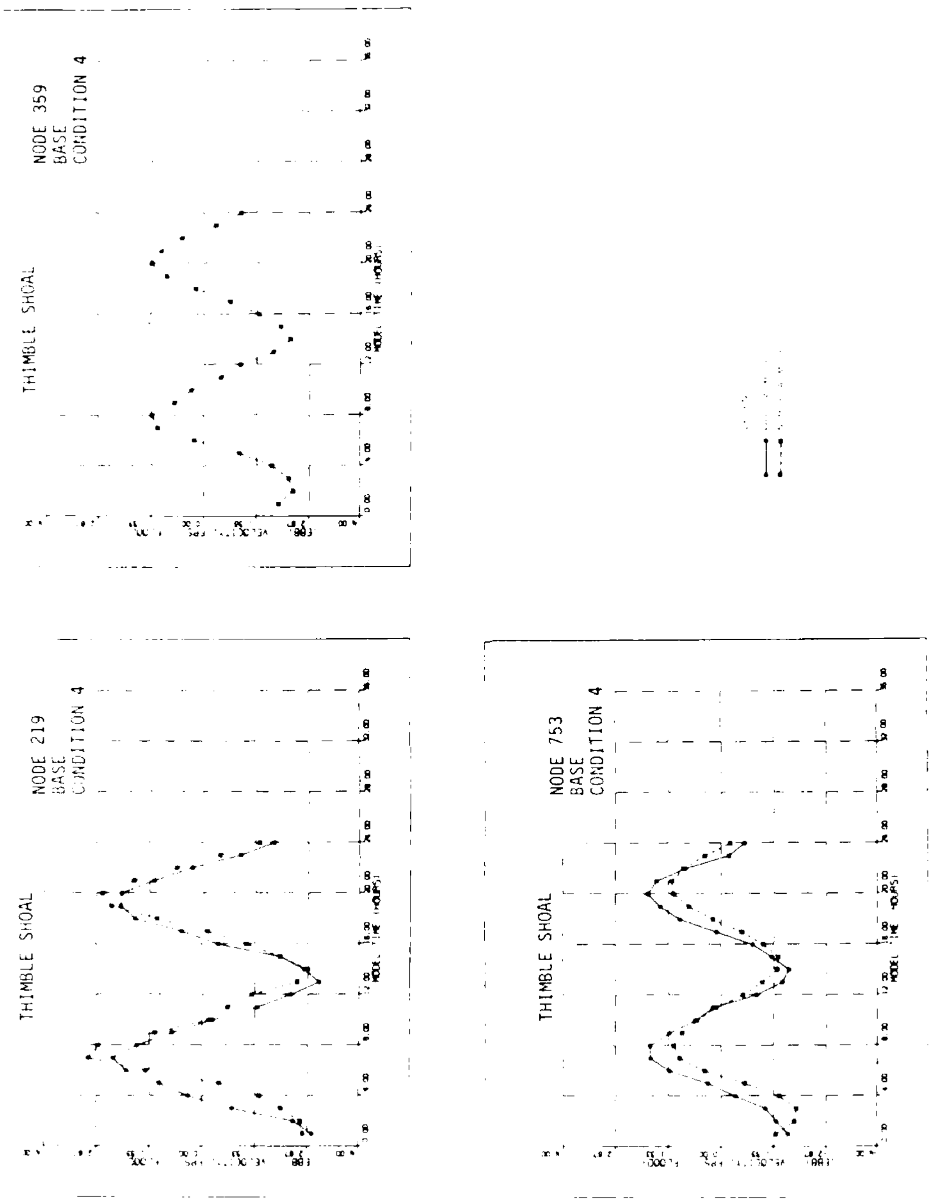
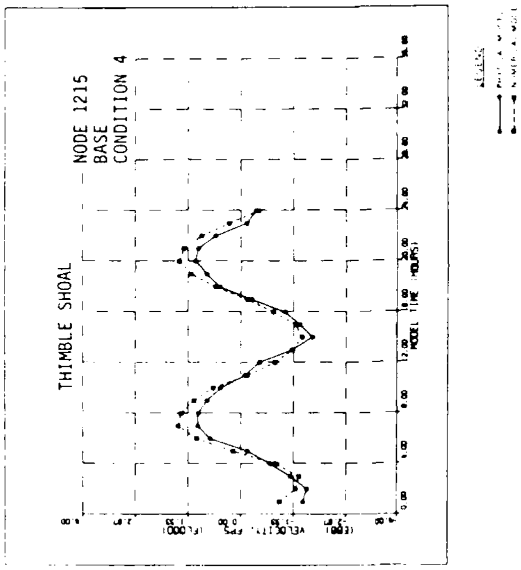

THIMBLE SHOAL
NODE 359
BASE
CONDITION 4
THIMBLE SHOAL
NODE 219
BASE
CONDITION 4
THIMBLE SHOAL
NODE 753
BASE
CONDITION 4

PLATE 35


THIMBLE SHOAL
NODE 1215
BASE
CONDITION 4
VELOCITY, FPS
MODEL TIME (HOURS)

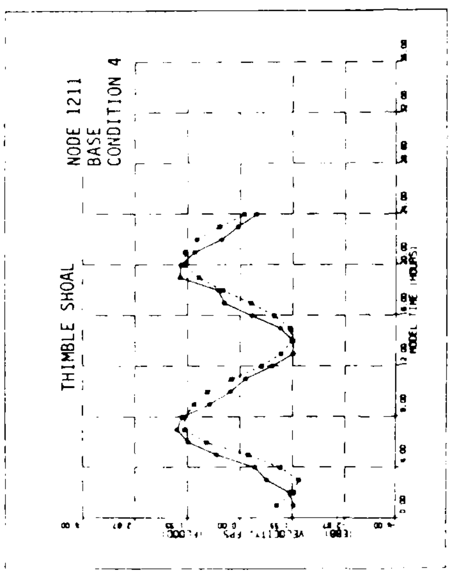
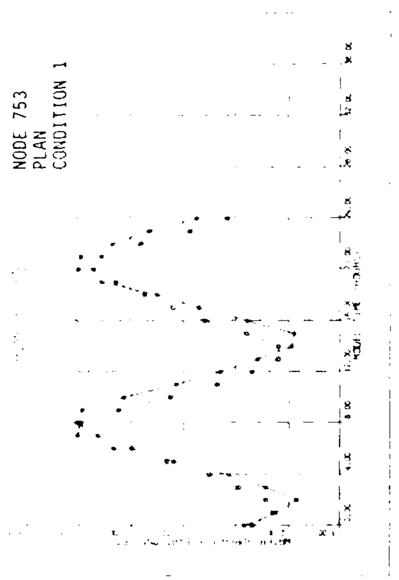
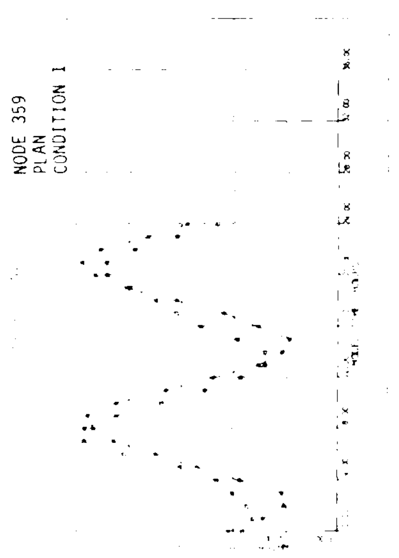

THIMBLE SHOAL
NODE 1211
BASE
CONDITION 4
VELOCITY, FPS
MODEL TIME (HOURS)

PLATE 36


NODE 753
PLAN
CONDITION 1


NODE 359
PLAN
CONDITION 1

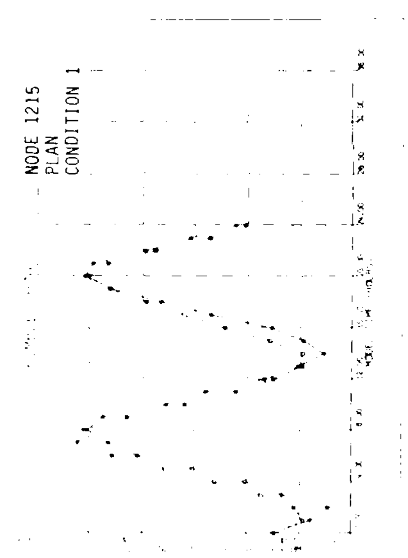
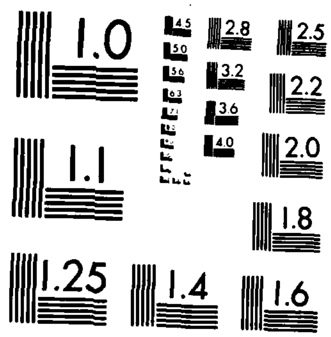
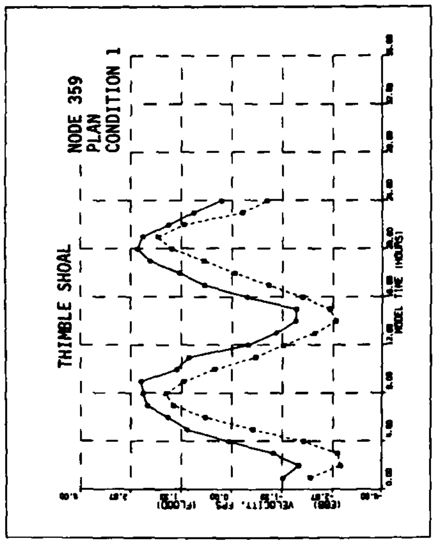

NODE 1215
PLAN
CONDITION 1

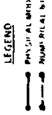
PLATE 37

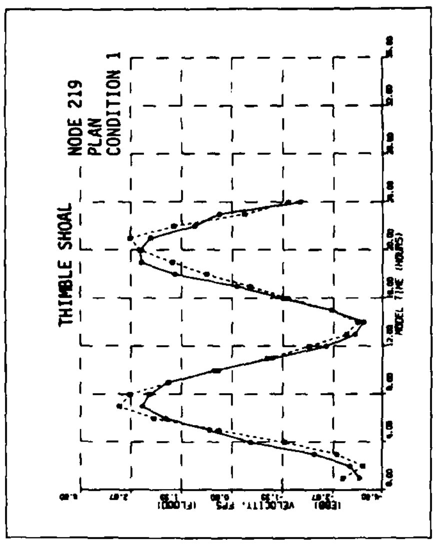
AD-A154 978 NORFOLK HARBOR AND CHANNELS DEEPENING STUDY REPORT 2 2/2
SEDIMENTATION INVEST..(U) ARMY ENGINEER WATERWAYS
EXPERIMENT STATION VICKSBURG MS HYDRA..
UNCLASSIFIED R C BERGER ET AL. MAR 85 WES/TR/HL-83-13-2 F/G 8/8 NL

END
FILMED
DTIC


1.0
2.8
2.5
3.2
2.2
3.6
4.0
2.0
1.1
1.8
1.25
1.4
1.6


THIMBLE SHOAL
NODE 359
PLAN
CONDITION 1
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)


LEGEND


THIMBLE SHOAL
NODE 219
PLAN
CONDITION 1
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)

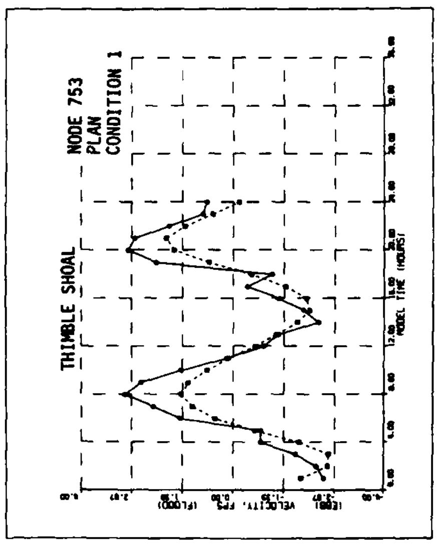

THIMBLE SHOAL
NODE 753
PLAN
CONDITION 1
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)

PLATE 38

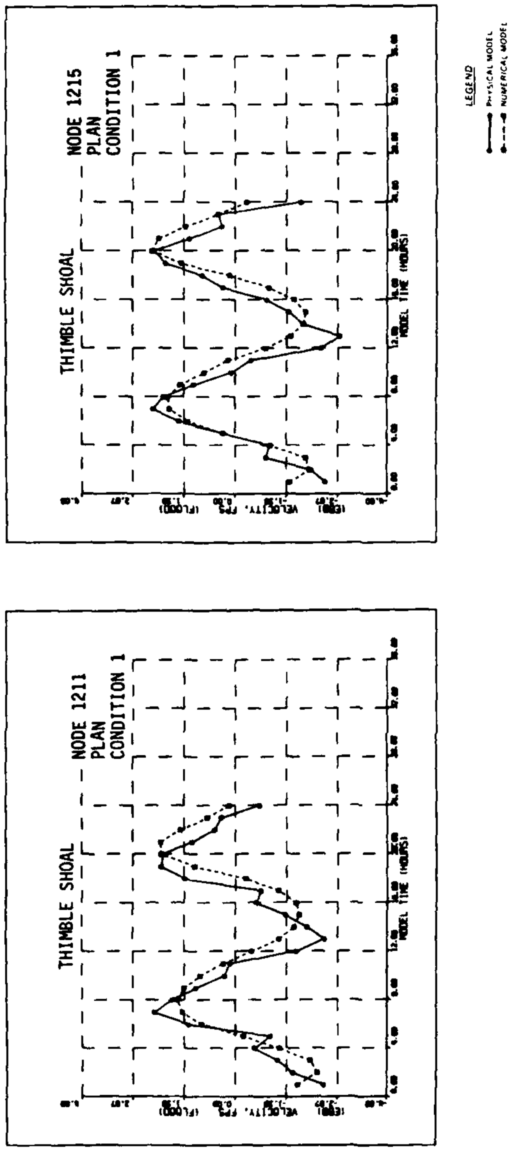

THIMBLE SHOAL
NODE 1211
PLAN
CONDITION 1
MODEL TIME (HOURS)
(EBB) VELOCITY, FPS (FLOOD)
THIMBLE SHOAL
NODE 1215
PLAN
CONDITION 1
MODEL TIME (HOURS)
(EBB) VELOCITY, FPS (FLOOD)
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL

PLATE 39

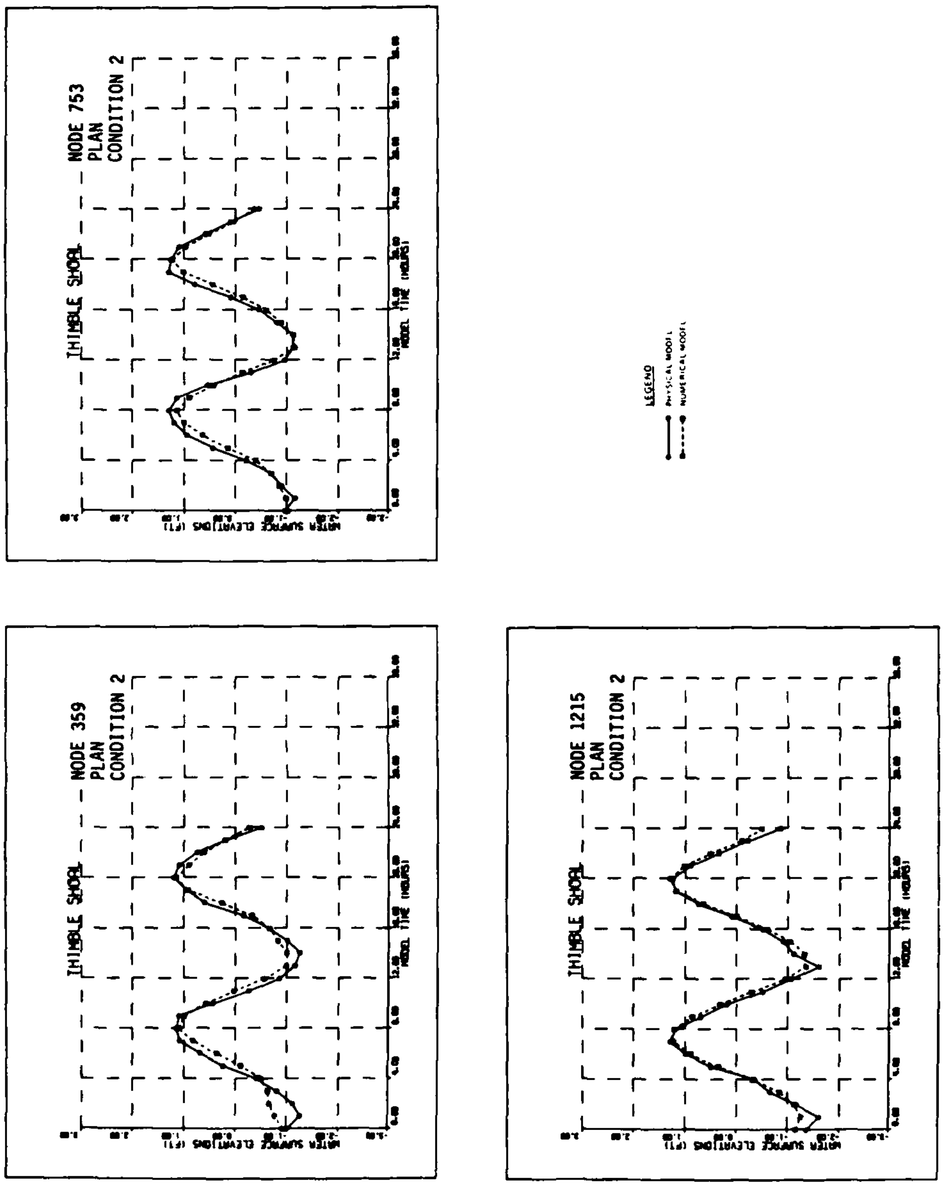
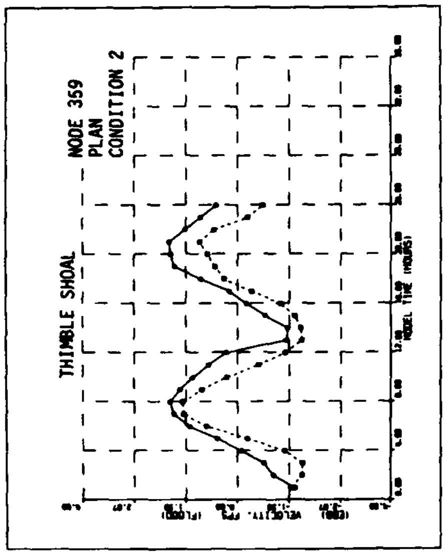
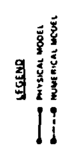
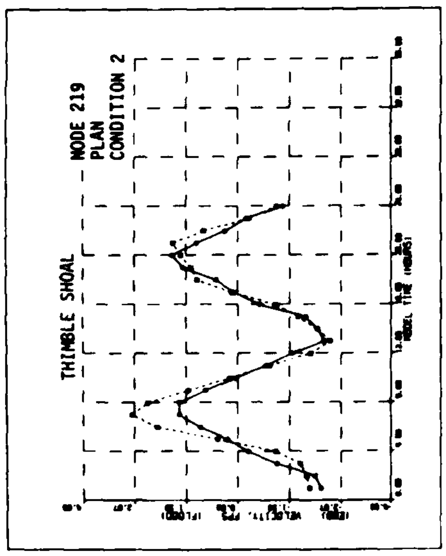

THIMBLE SHOAL NODE 753 PLAN CONDITION 2
THIMBLE SHOAL NODE 359 PLAN CONDITION 2
THIMBLE SHOAL NODE 1215 PLAN CONDITION 2
LEGEND

PLATE 40


THIMBLE SHOAL
NODE 359
PLAN
CONDITION 2
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)


LEGEND
PHYSICAL MODEL
NUMERICAL MODEL


THIMBLE SHOAL
NODE 219
PLAN
CONDITION 2
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)

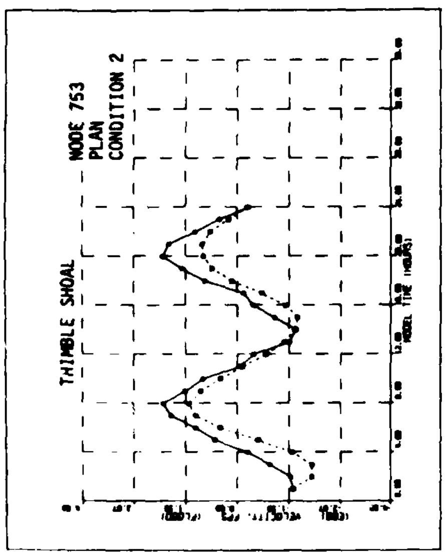
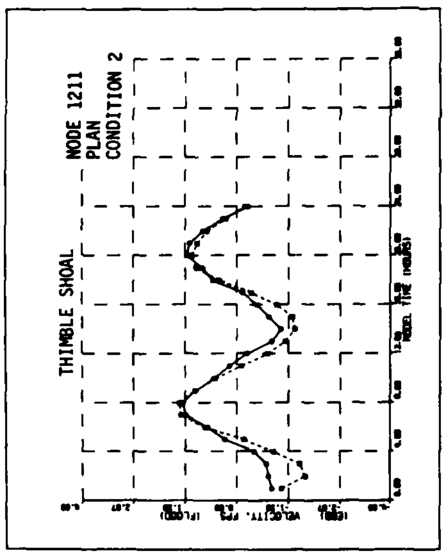

THIMBLE SHOAL
NODE 753
PLAN
CONDITION 2
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)

PLATE 41


THIMBLE SHOAL
NODE 1211
PLAN
CONDITION 2
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)

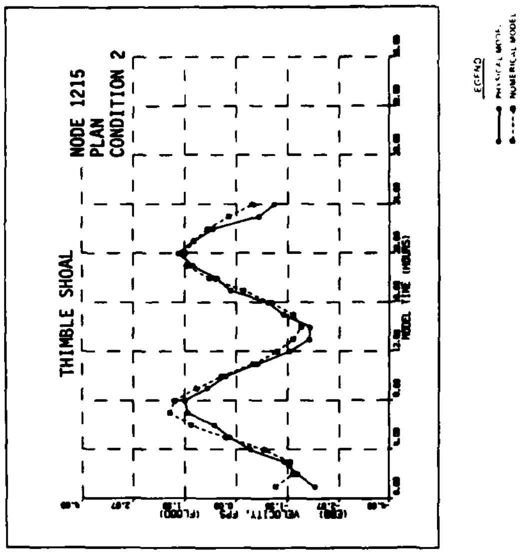

THIMBLE SHOAL
NODE 1215
PLAN
CONDITION 2
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL

PLATE 42

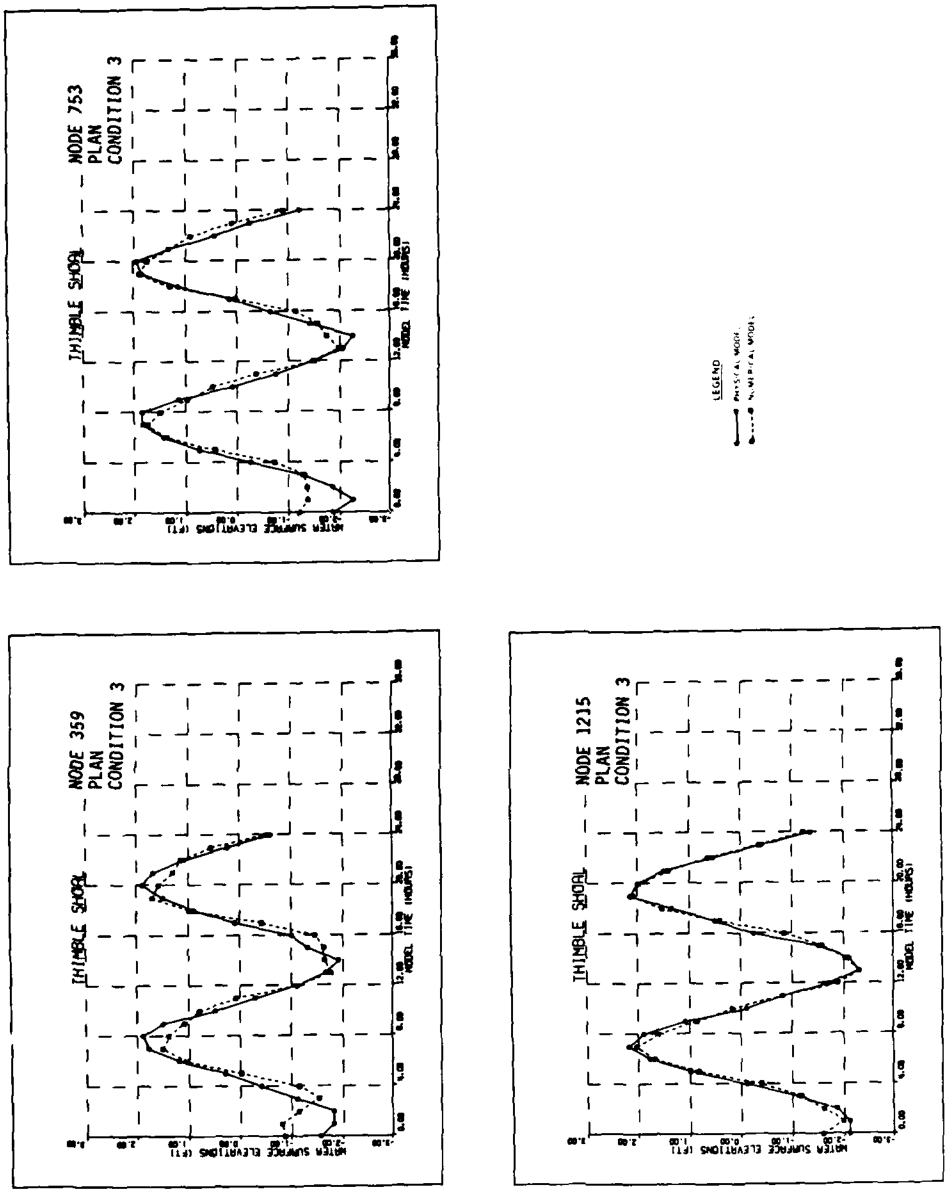

THIMBLE SHOAL
NODE 753
PLAN
CONDITION 3
THIMBLE SHOAL
NODE 359
PLAN
CONDITION 3
THIMBLE SHOAL
NODE 1215
PLAN
CONDITION 3
WATER SURFACE ELEVATIONS (FT)
MODEL TIME (HOURS)
LEGEND

PLATE 43

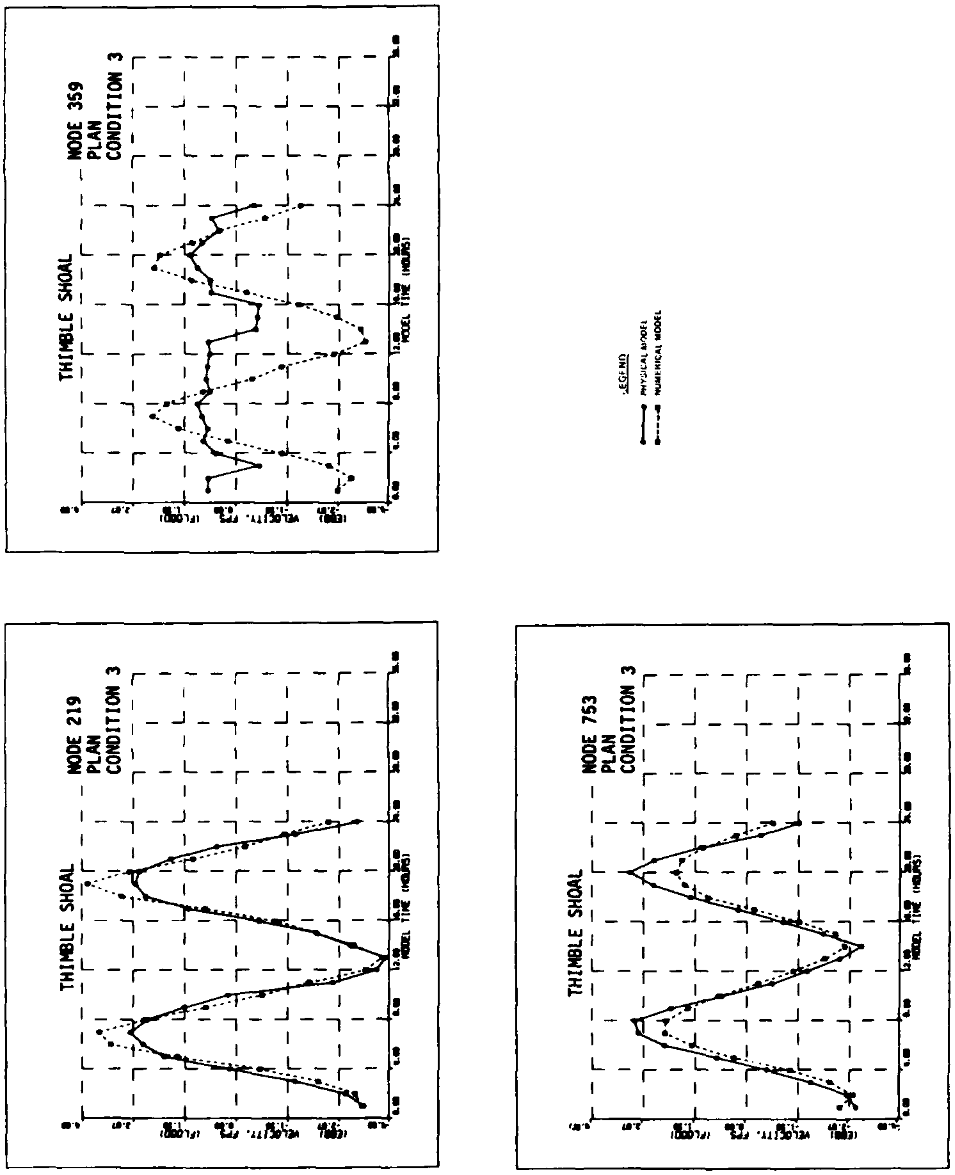

THIMBLE SHOAL
NODE 359
PLAN
CONDITION 3
THIMBLE SHOAL
NODE 219
PLAN
CONDITION 3
THIMBLE SHOAL
NODE 753
PLAN
CONDITION 3
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL

PLATE 44

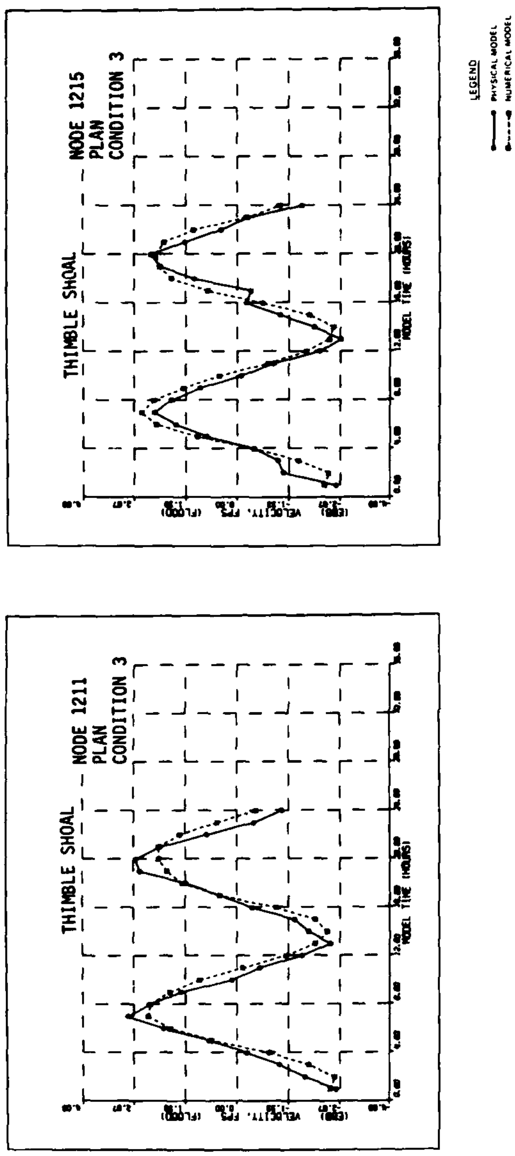

THIMBLE SHOAL
NODE 1211
PLAN
CONDITION 3
THIMBLE SHOAL
NODE 1215
PLAN
CONDITION 3
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL

PLATE 45

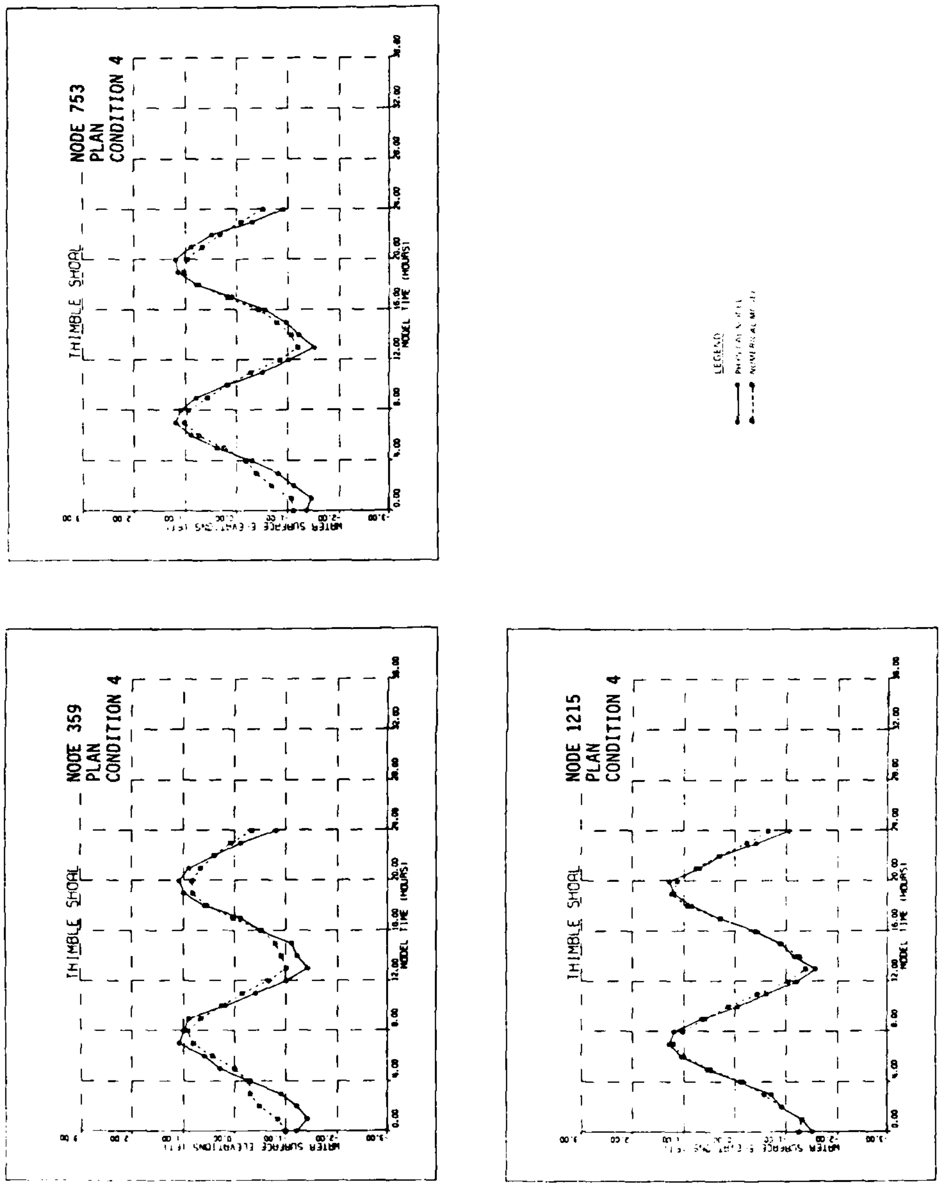

THIMBLE SHOAL
NODE 753
PLAN
CONDITION 4
THIMBLE SHOAL
NODE 359
PLAN
CONDITION 4
THIMBLE SHOAL
NODE 1215
PLAN
CONDITION 4
MODEL TIME (HOURS)
WATER SURFACE ELEVATIONS (FT)
LEGEND

PLATE 46

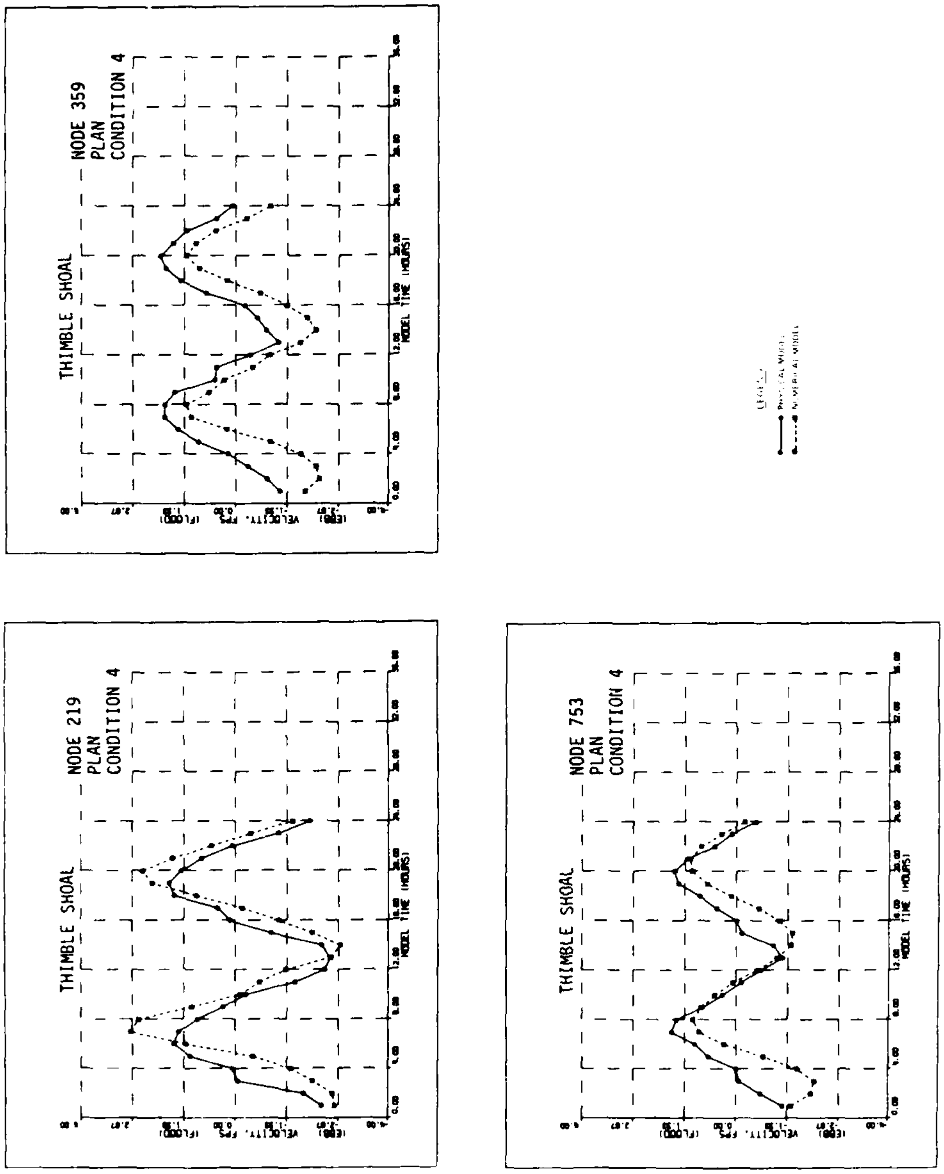

THIMBLE SHOAL
NODE 359
PLAN
CONDITION 4
THIMBLE SHOAL
NODE 219
PLAN
CONDITION 4
THIMBLE SHOAL
NODE 753
PLAN
CONDITION 4
MODEL TIME (HOURS)

PLATE 47

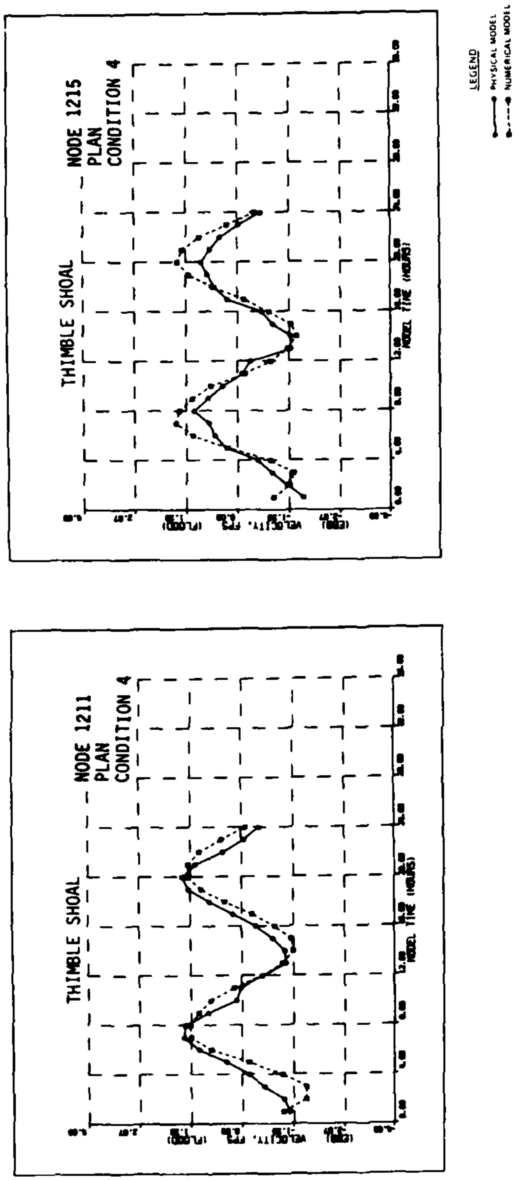

THIMBLE SHOAL
NODE 1215
PLAN
CONDITION 4
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)
LEGEND
PHYSICAL MODEL
NUMERICAL MODEL
THIMBLE SHOAL
NODE 1211
PLAN
CONDITION 4
VELOCITY, FPS
(FLOOD)
(EBB)
MODEL TIME (HOURS)

PLATE 48

APPENDIX A: ATLANTIC OCEAN CHANNEL SEDIMENTATION PREDICTIONS

Introduction

ground

1. The development of the Atlantic Ocean Channel is part of the ıosed improvements to channels approaching Norfolk Harbor and anchorages. ; new channel, located off Virginia Beach, is to be 57 ft below mean water and have a width of 1,000 ft over a length of 10.6 statute miles. ; stated depth of the channel is the project depth. The actual depth the proposed new channel with the dredging tolerance and advance main- ınce will be 60 ft below mean low water. Figure A1 shows the project ıils.

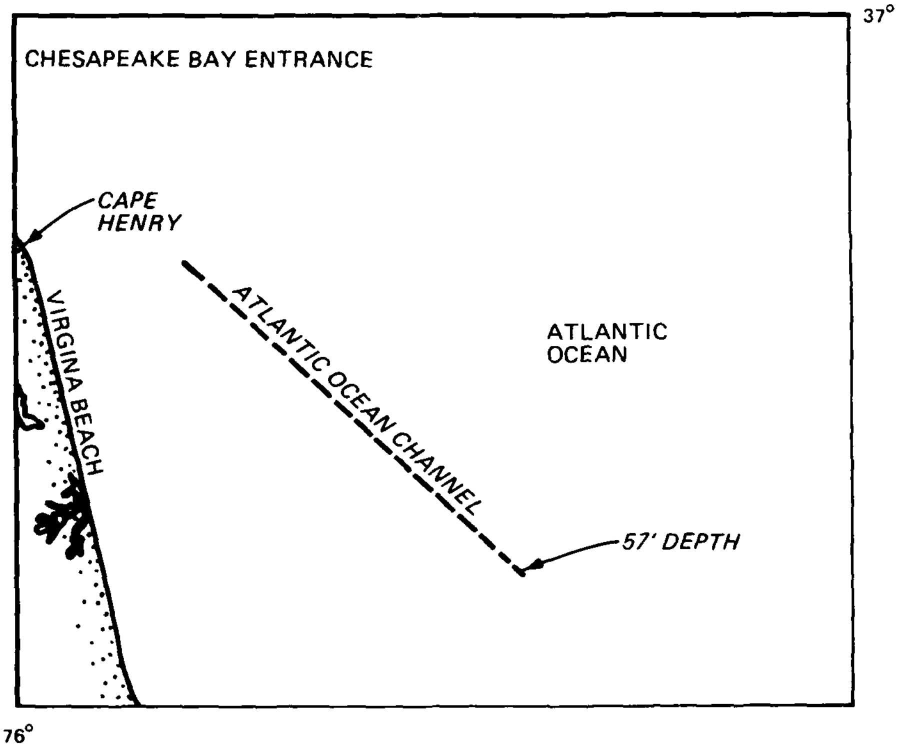


Figure A1. Project location

pos

2. The purpose of this portion of the broader study was to analyti- ly or empirically predict expected shoaling rates in this channel once it developed. This was to be accomplished using data that were available.

e greater than 30 cm/sec 9.30 percent of the time. For the 60-ft Atlantic ean Channel the wave orbital velocities are greater than 30 cm/sec 8.39 per-nt of the time. Therefore the ability of the waves to at least place fine nd into suspension would only be changed by approximately 1 percent with e deepened channel.

'drographic survey comparisons

32. At the site of the proposed Atlantic Ocean Channel three hydro-'aphic surveys can be compared to investigate gross changes in bathymetry. ıe 1947 survey is a patchwork with many areas of no data. The 1980 survey ıcluded the entire study area. The 1969 survey did not extend past 75°49' .ongitude) and 36°51' (latitude); therefore only the first 7.5 miles of the ıannel can be compared.

33. Both the 1969 and 1980 survey data were collected by an on-line 'quisition system (Gardner 1982*) and referenced to mlw.

34. To compare surveys, comparison points were located on the hydro-raphic surveys and the 10 depths surrounding this point were recorded and veraged. From the 23 comparisons made between the 1969 to 1980 data there as a natural deepening or erosion that averaged about 20 mm/yr. From the 947 to 1969 surveys the computed erosion averaged about 120 mm/yr.

35. The 1969 and 1980 urveys were compared in more etail using a computer-based ystem developed at the US rmy Engineer Waterways Ex-eriment Station (LaGarde and eltzel 1980). Figure A9 hows the coverage of the urveys compared. The 1969 urvey sheets were reduced to he scale of the 1980 sheets. oth surveys were contoured by and and then digitized and

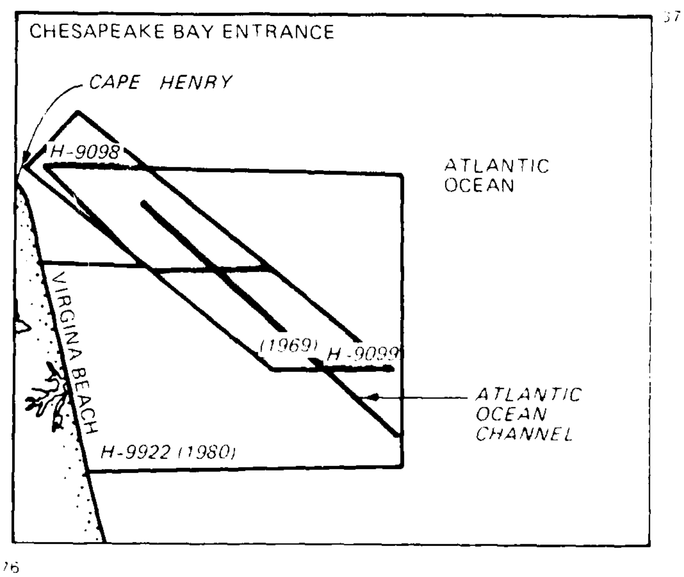


Figure A9. Hydrographic survey coverage

Personal communication, LT J. C. Gardner, Jr. (1983), NOAA, Norfolk, Va.

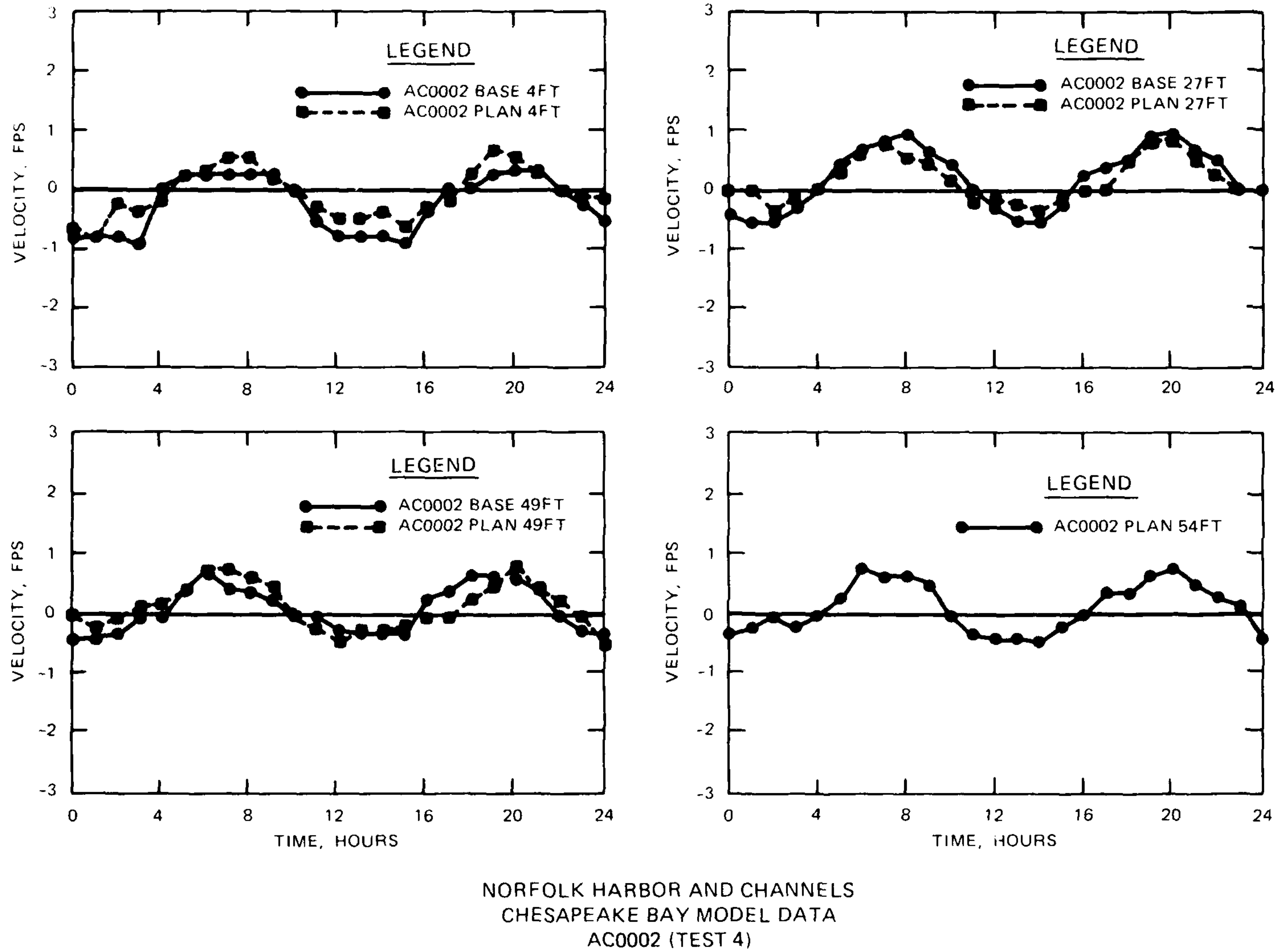
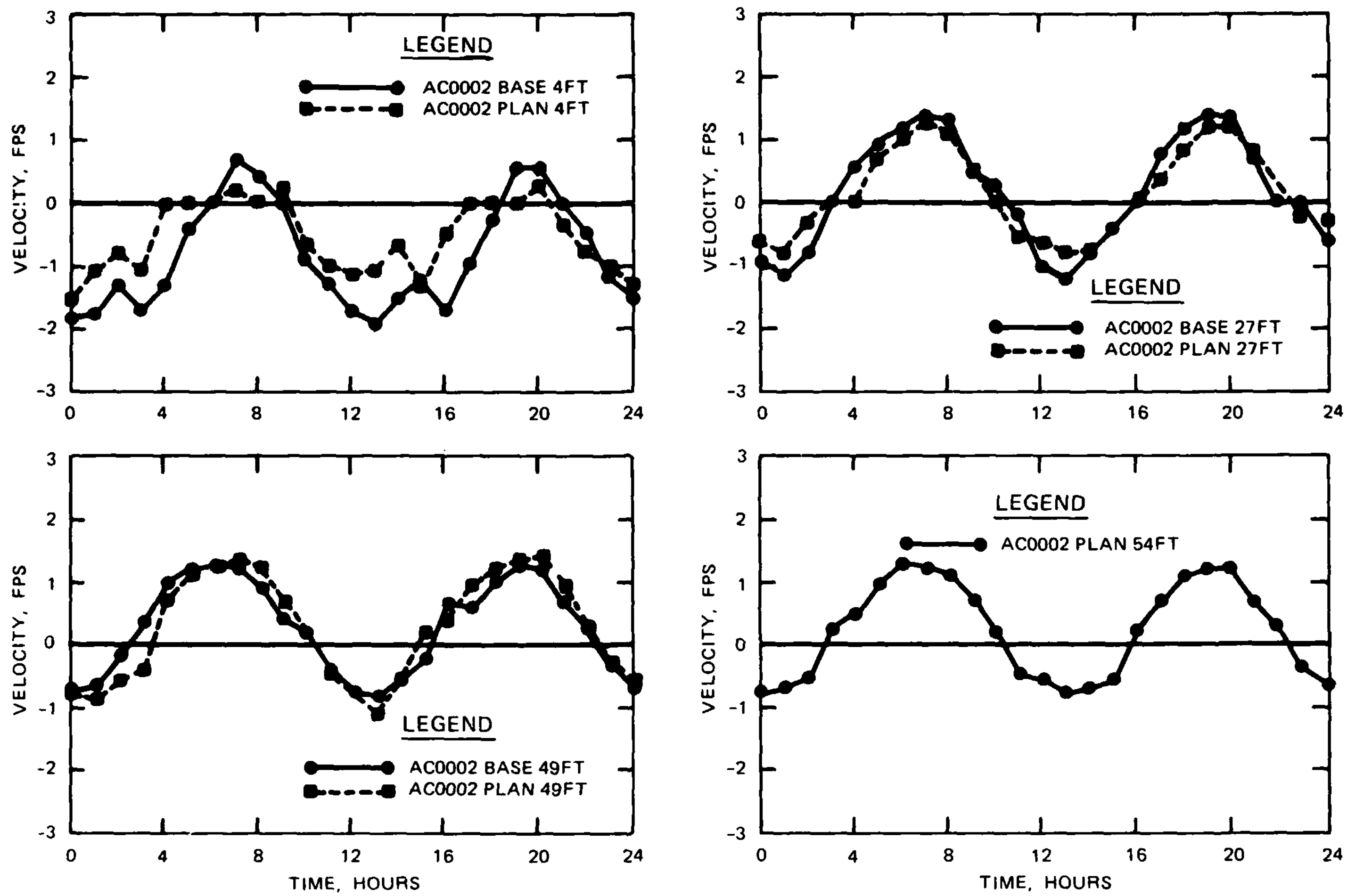


Figure A8. Chesapeake Bay model data (Test 4)



NORFOLK HARBOR AND CHANNELS
CHESAPEAKE BAY MODEL DATA
AC0002 (TEST 3)

Figure A7. Chesapeake Bay model data (Test 3)

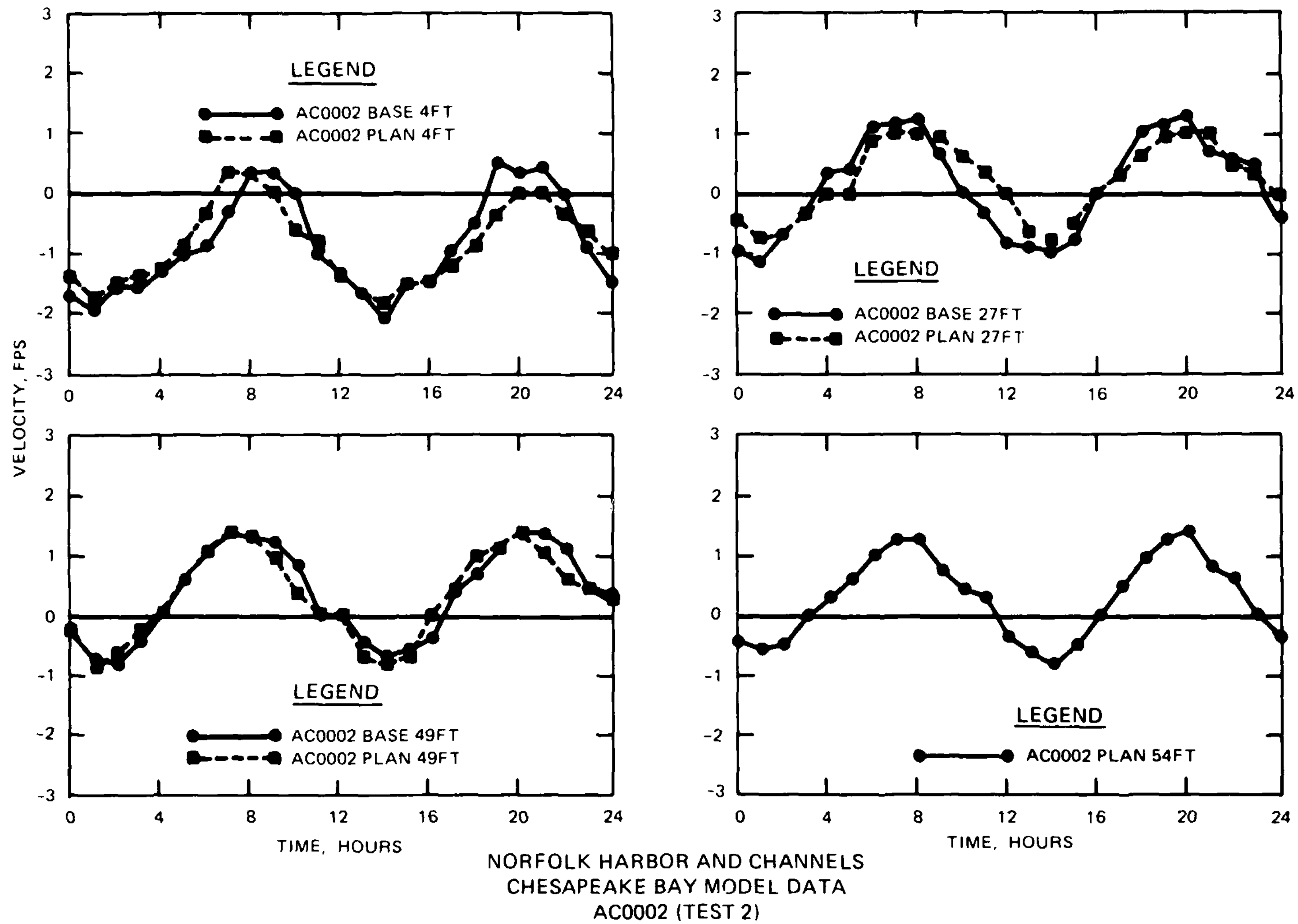
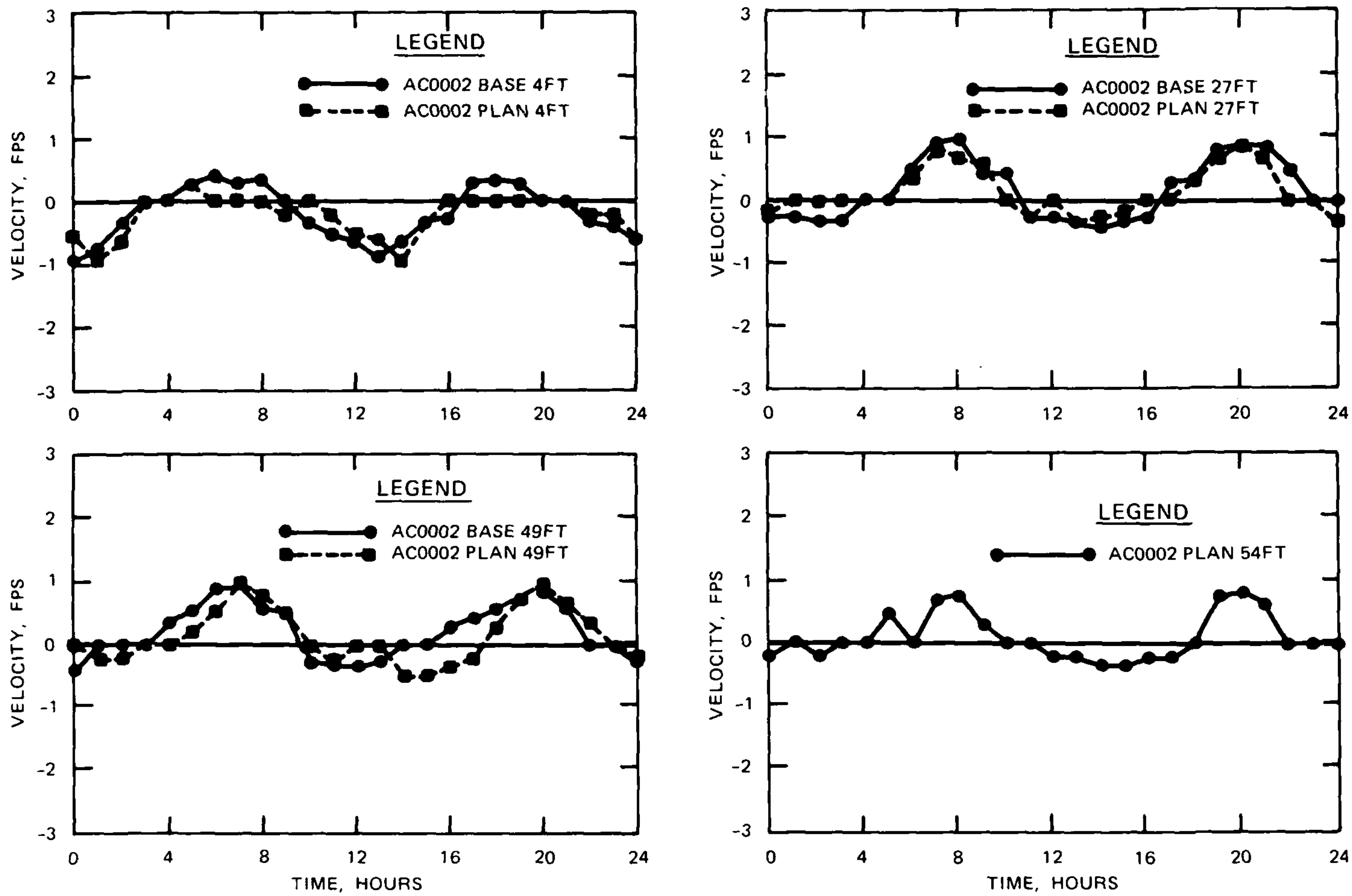


Figure A6. Chesapeake Bay model data (Test 2)



NORFOLK HARBOR AND CHANNELS
CHESAPEAKE BAY MODEL DATA
AC0002 (TEST 1)

Figure A5. Chesapeake Bay model data (Test 1)

Condition 2: The total bay discharge remained at 200,000 cfs; however, the repetitive cosine tide range was changed to ±1.50 ft, which approximates a neap tide condition.

Condition 3: The total bay discharge was 70,000 cfs, which simulates the long-term average flow into the Chesapeake Bay from all its tributaries. The approximate spring tide range of ±2.4 ft was generated at the ocean.

Condition 4: The total bay discharge remained at 70,000 cfs and the approximate neap tide range of ±1.5 ft was generated at the ocean.

28. Velocity plots (Figures A5-A8) of the data indicate flood predominance at all depths except the surface and this pattern existed during all the test conditions (flood values are positive). An analysis of direction data indicated that during a majority of the four test conditions the flow was along the axis of the channel. The spring tidal range conditions produced a few velocities greater than 1 fps; however, during the neap tidal range, all velocities were less than 1 fps. This velocity would be the minimum required to move sediment found in the study area from the bed into suspension. Results from the test with the Atlantic Ocean Channel installed indicated the same general trends but the velocities were reduced. Therefore tidal currents alone would not be sufficient to cause any significant change in the bed.

Wave climatology study for Chesapeake Bay entrance and Virginia Beach

29. Wave climatology statistics of the Chesapeake Bay entrance and Virginia Beach areas for the 20-year period from 1956-1975 are given in Tables A1 and A2, respectively (Jensen 1982*). These wave statistics (significant waves in height and period) include 20-year summaries (percent occurrence) for the entire period for all directions.

30. From the 20-year summary of wave statistics for Virginia Beach, it is observed that from 58,440 significant waves, 81 percent had a height of 1.49 m or less and 71.15 percent of the periods were 7.9 sec or less. The largest significant wave was 5.35 m and the average was 0.55 m.

31. Using the method outlined by Beauchamp (1974) to determine the maximum orbital velocity, it was determined that at the present average depth of the location of the proposed channel (56.96 ft) wave orbital velocities

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* Personal communication, R. E. Jensen, 1982, Wave Information data base, provided by Coastal Engineering Research Center, USAEWES.

dredged channel, located at Port Said Harbor, Egypt. In the method outlined, an average steady current characterized by a velocity and a wave height and period is required. From this information a maximum oscillatory current at the bed due to wave action is calculated. Values are obtained for the two components of sediment transport (bed and suspended load), based on the maximum oscillatory current and selected descriptive sediment and water parameters, in addition to other laboratory derived variables. In estimating the deposition rate, it is assumed that part of the suspended load and all of the bed load are deposited within the channel. The change in deposition is based on the changes in the load-carrying capacity. Thus the deposition is the sum of the bed load and difference between the suspended load approaching and leaving the channel.

26. Ludwick (1981) proposed a short method that produces reasonable results. He simplifies the equation for deposition of fine-grained sediment in the presence of a moving tidal current by assuming that for a specified short period of time, the shear stress at the bed is substantially less than the critical shear stress for deposition. Thus the mass of sediment deposited per unit time is equal to the average concentration times an average settling velocity. A thickness rate of deposition is determined from the mass of sediment deposited by estimating an in-place bulk density of the sediment. The deposition period referred to is taken as a 1-hr period at slack water before ebb and at slack water before flood. This assumption results in deposition only occurring 2.0 hr out of every 12.42 hr.

Results

Chesapeake Bay physical model data analysis

27. Velocity measurements were made in the Chesapeake Bay physical model at a station east of Rudee Inlet and approximately in the center of the proposed Atlantic Ocean Channel. This station is designated as AC0002. This was done to examine the impact of channel deepening on current velocities. The following were the four steady-state boundary conditions:

Condition 1: The model was operated under a repetitive cosine tide having a range of ±2.40 ft at the Atlantic Ocean control station. This approximates a spring tide condition in Chesapeake Bay. The total bay freshwater river inflow was a constant 200,000 cfs, which represents a relatively high flooding condition.

Available analytical and empirical methods

21. Many techniques have been used in the past to predict the effects of channel enlargement on channel shoaling. These range from rule-of-thumb predictions to elaborate physical model investigations. During the past few years, there has been progress in the use of numerical models or combined physical and numerical models (hybrid studies) for making these predictions. Empirical techniques and analytic solutions have been used extensively and provide valuable easily obtained solutions; they are, in most cases, scientifically based.

22. Trawle* has done a thorough review of some of the available analytic and empirical methods for estimating infill rates in offshore channels. These techniques can be used when consideration is being given to a channel enlargement or to the use of advance maintenance.

23. Trawle* describes four analytically based and three empirically based methods. He identifies the four analytic methods as the Moriches Inlet method, the Lean method, the Oregon Inlet method, and the Lamblé method. The three empirical methods discussed are identified as the Gole method, the Simplified Shoaling Rate method, and the Volume of Cut method. These represent some of the better documented techniques currently being used.

24. Boicourt (1981) uses what may be termed a single-grain characterization. This technique is implemented by collecting near-bottom velocity data and determining a mean grain size for the sediment in the area. The current meter data 3 m or so above the bottom are assumed to be appropriate for a conservative estimate of the threshold velocity for initiation of grain movement. A threshold velocity for initiation of grain movement for that particular sediment size is calculated. Then he determines from his velocity record how much of the time this value is exceeded. By comparing these data he makes a qualitative assessment of when sediment transport events are likely to occur. This comparison could be done seasonally.

25. Kadib (1976) describes yet another method to predict sedimentation at offshore dredged channels. His approach is based on theoretical studies and experiences gained with maintenance dredging at the Suez Canal offshore

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* M. J. Trawle. Engineer Technical Letter 1110-2-293, Engineering and Design, "Shoaling Predictions in Offshore Navigation Channels, Analytical and Empirical Methods."

produce a net sediment transport. Actual near-bottom velocities have a larger forward velocity of shorter duration under the wave crests and a smaller backward velocity of longer duration under the troughs. A net sediment transport can be caused by this small difference between large quantities of sediment in motion. Laboratory observations and field work have confirmed that sediment transport outside the breaker zone is composed of both bed and suspended load. The near-bottom shear velocity is the same order of magnitude as the fall velocity of the sediment particles.

19. Hales (1980) discusses in detail the combined effects of current and wave motion. He notes that waves change characteristics as they progress from relatively calm water into regions of streaming water. Waves traveling with the current experience an increase in length and celerity and a decrease in height. Waves traveling against the current increase in height and decrease in length and celerity until a limiting steepness occurs that depends both on the initial wave characteristics and the strength of the opposing current (Hales 1980).

Review of Potential Solution Techniques

Introduction

20. In the WESHE proposal dated 20 August 1981, Subject: Time and Cost Estimates for Norfolk Harbor 55-Foot Channel Study, a plan of study for the Atlantic Ocean reach was proposed: "The Atlantic Ocean area of the Chesapeake Bay model has not been verified to properly reproduce prototype currents; therefore the Atlantic Ocean Channel is not amenable to a hybrid modeling technique employing the bay model. It is possible to perform a sedimentation study of that area by means of a completely numerical modeling effort, including modeling of wind waves and longshore currents, but is considered inadvisable because the cost (including extensive field data collection) does not appear to be justified by the magnitude of the shoaling problem in that channel. Instead we recommend an analytic study using available data to predict an expected channel shoaling rate. Presently, we have an active research study under way to develop the most effective procedures to address sediment predictions in entrance channels. The most effective technique resulting from this research effort will be used to investigate this portion of the study."

Theoretical Background of the Problem

14. A thorough treatment of the problems of unsteady oscillatory flow including threshold of sediment motion under wave action and combined current and wave motion is given by Hales (1980); these constitute PARTS IV and VI of Report 2. Concepts and ideas that are required for understanding siltation in ocean areas are discussed below.

15. Boundary shear stress in sediment transport problems is of primary importance. Hales (1980) states that oscillatory fluid motion associated with surface gravity waves exerts shear stresses on the bottom that are often several times larger than shear stresses produced by unidirectional currents of the same magnitude. He further states that shear stresses produced by wave motion may put sediments into suspension where they can be transported by currents of a magnitude insufficient to initiate sediment motion.

16. In riverine environments the threshold for the initiation of sediment motion has been reasonably well documented. However, this is not the case for sediment movement under oscillatory wind waves. This is primarily related to the difficulty of taking measurements under these conditions. Most researchers in this area apply the curves of Shields for unidirectional flow to the wave condition considering that there might be some error since these accelerating currents exert a larger shear stress than does a steady flow of the same magnitude. The evidence is overwhelming that the Shields function with its limitations serves as a reliable and quite general criterion for the threshold of sediment movement under water waves.

17. Komar and Miller (1974) developed relations between threshold of grain movement defined by a grain diameter d and density ρ_s and wave period T and the orbital velocity at the near-bottom u_{max}. All the data used in this development came from laboratory experiments. Another consideration stressed in the review by Hales is that the interaction of wave trains of differing periods under natural conditions may generate instantaneously higher velocities, and sediment motion may occur at lower velocities than those implied in this analysis.

18. Oscillatory flow produces complex flow patterns that vary continually in magnitude and direction. Linear wave theory describes this condition as one with no net transport. However, various nonlinear effects, such as wave asymmetry, and wave-induced net transport, modify this equilibrium and

to easterly waves from offshore and to waves generated within the lower bay which may reach heights of over 4 ft, especially with northerly winds.

Sediment distribution

13. The dominant surficial sediment of Chesapeake Bay entrance is a homogeneous gray, fine to very fine quartzose sand, usually well-sorted and often silty (Meisburger 1972). Figure A4 shows the surface sediment distribution in the study area.

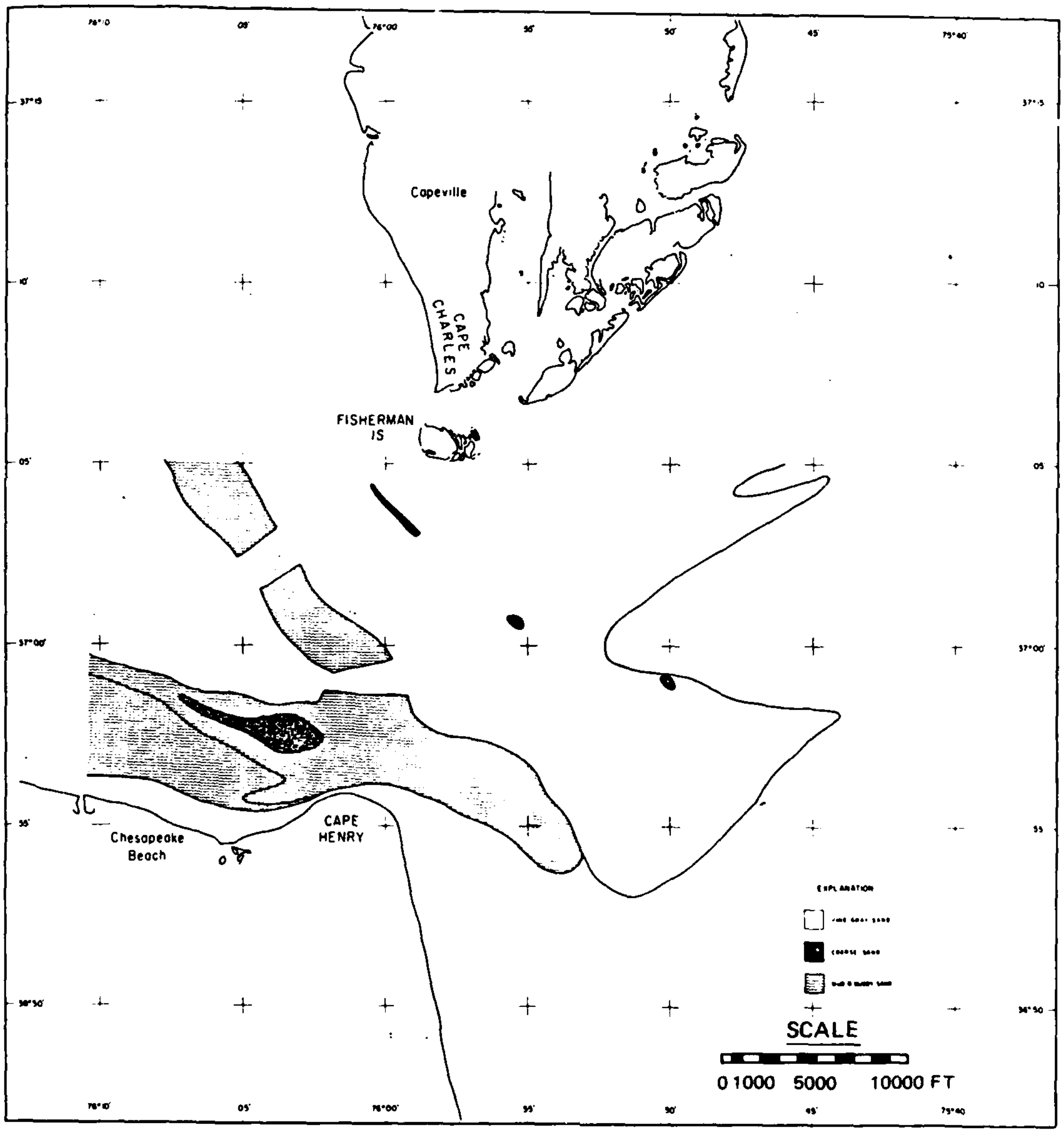

Figure A4. Surface sediment distribution in the study is based on gross characteristics (Meisburger 1972)

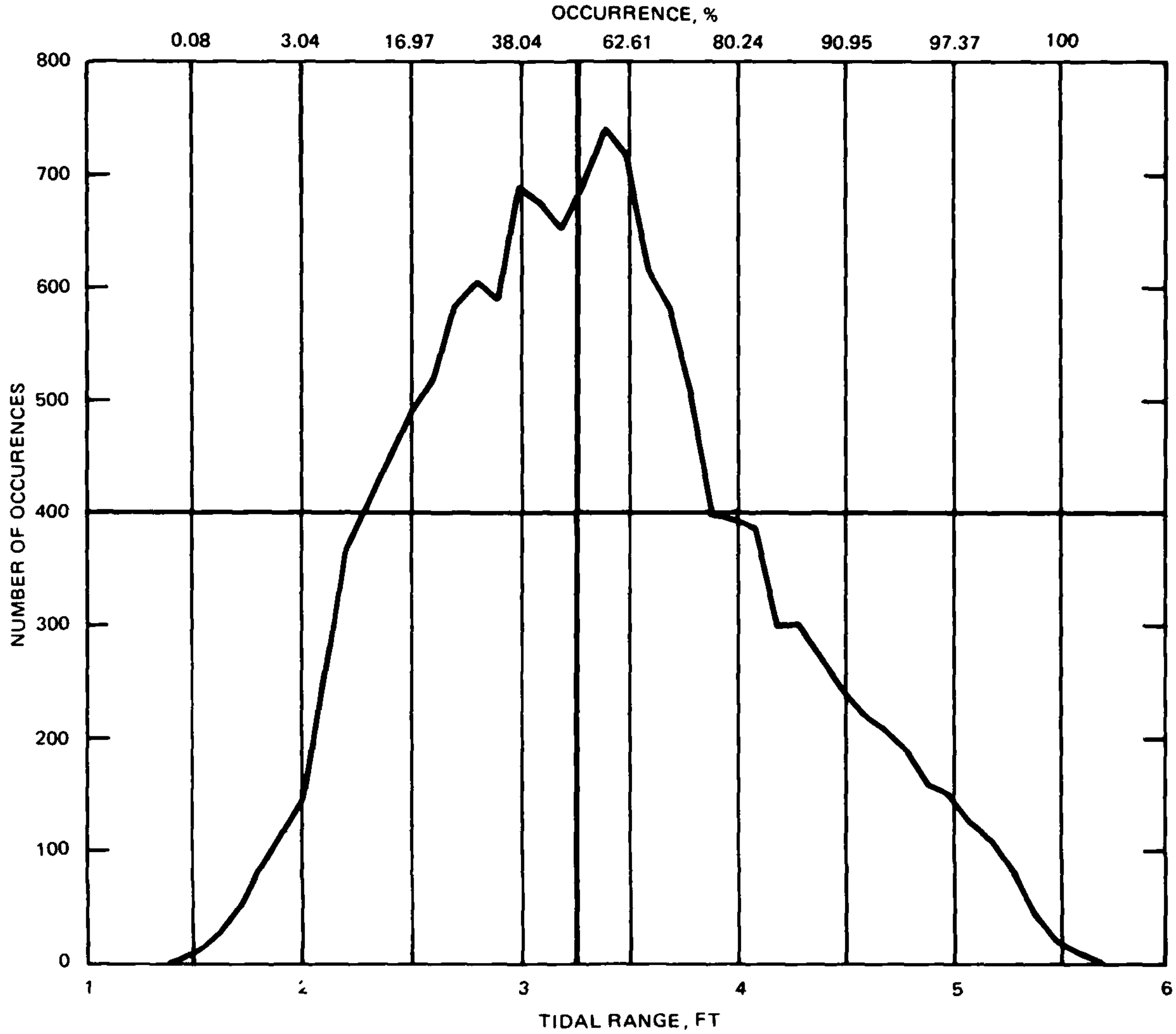

Figure A3. Tidal range, ft, 20-year tidal range statistics, Virginia Beach, Virginia

10. Bosserman and Dolan (1968) studied 857 storms for the period 1942-1967 and constructed a storm probability curve for three hindcast deep-water wave-height intervals. During the Ash Wednesday storm of March 1962, the maximum computed significant wave height was about 10 m. The severest storms occur from January through March.

11. Hurricanes generally move from southwest to northeast in the study area (Ho and Tracey 1975). The number of hurricanes affecting the area increases from Cape Henry to Cape Hatteras.

12. Waves on the open coast south of Cape Henry as measured by a Coastal Engineering Research Center (CERC) wave gage at Virginia Beach are less than 3 ft high more than 90 percent of the time. Most of the bay entrance is open

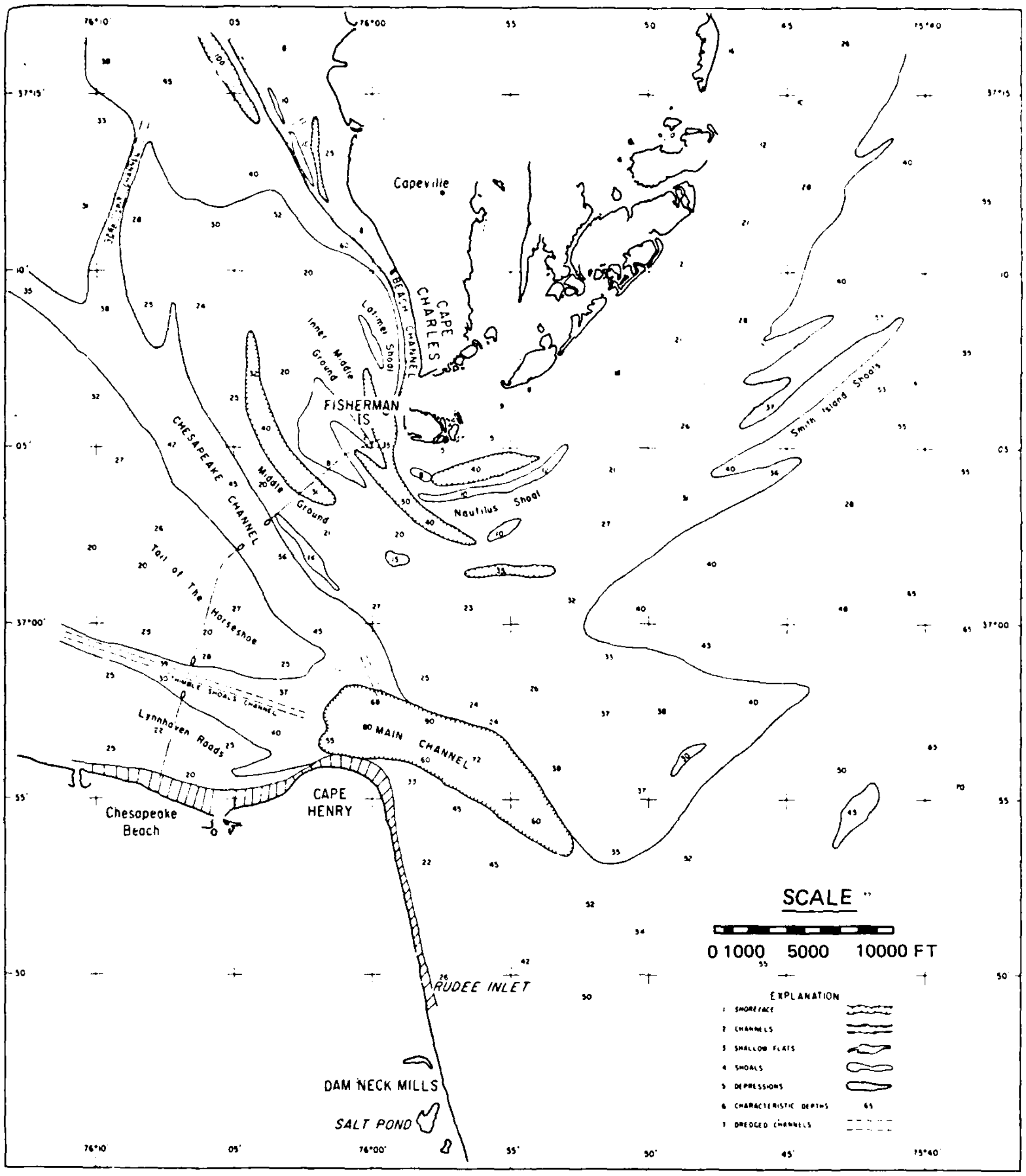

Figure A2. Gross morphology of the bottom in Chesapeake Bay entrance area. Soundings are in feet (Meisburger 1972)

relative to land at Norfolk, Virginia, was +4.4 mm/yr. From 1940 to 1978, the average was +3.7 mm/yr, or about 15 percent less than the 1928-1978 average.

Meteorological and wave influences

9. Mean annual wind velocities are 16 km/hr at Cape Henry. Winds at or above this speed are predominantly onshore from the northeast and occur most frequently during the winter months.

Fluid Dynamics of the Chesapeake Bay Entrance

Morphology

3. A major portion of the Chesapeake Bay entrance is less than 35 ft in depth, but greater depths occur in the channels where there are a few isolated closed depth depression contours.

4. The entrance is 10 miles across and extends from Fisherman Island near Cape Charles to Cape Henry; however, the main natural inlet channel is less than 2 n.m. in width. This channel is deepest off Cape Henry and extends southeastward for about 5 n.m. The Tail of the Horseshoe is a sandy shoal between the Thimble Shoal and Chesapeake Channels. The Cape Charles terrace is characterized by numerous secondary morphological features, among which linear shoals and semiclosed depressions are most common (Meisburger 1972*). Figure A2 shows the gross morphology of the bottom in the Chesapeake Bay entrance.

Entrance and shelf circulation

5. The circulation in the Chesapeake Bay is primarily a result of tidal action and wind.

6. Net nontidal circulation in the Chesapeake Bay Bight (Cape Henlopen, Delaware to Cape Hatteras) was documented by Harrison, Brehmer, and Stone (1964). Their bottom drifters released on the shelf up to distances as much as approximately 40 n.m. offshore tend to drift shoreward with some even having a tendency to travel toward and enter the Chesapeake Bay.

7. The tides in the Chesapeake Bay entrance are semidiurnal with a mean range of approximately 3 ft and a spring range of 3.5 ft. On the outer coast of Virginia, adjacent to the bay entrance, mean and spring ranges are about 3 and 4 ft, respectively. Although the magnitude of tidal currents will vary, they are generally in the range of 1 to 2 knots maximum. Figure A3 is the distribution of predicted tidal ranges for Virginia Beach summarized for the 20-year period from 1 January 1956 to 31 December 1975.

8. Sea-level change data are not available in the study area. However, tide gage records from Norfolk, Virginia, for the period 1928-1978 provide some information on the magnitude of changes that have occurred at the nearest stations (Hicks 1981). From 1928 to 1978, the average rate of sea-level rise

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* See References at end of main text.

gridded by automated techniques. A section map of the area was also made that determined regions where the surveys would be compared (Figure A10).

36. The algorithm calculates a surface using an interpolative procedure based on input data of nearest neighbors to the calculation location, and the calculation procedure provides an exact fit to the input data at all input data locations.

37. Tables A3 and A4 summarize the calculated information from each section. Figure A11 shows the depth change (ft) in each section and Figure A12 shows the contour plots of the two surveys. Figure A12 does indicate a trend of natural deepening of the area where the proposed Atlantic Ocean Channel is to be built.

Analytic methods

38. The Moriches Inlet method was the only purely analytic method that could be applied to this study. In this method, a transport ratio is calculated. The expression used compares the sediment transport potential in the dredged cut with the sediment transport on the bar before dredging. For this application a transport ratio of 0.8785 was obtained which means the proposed channel would cause about a 12 percent reduction in sediment transport capability along the channel. In the Lean method, some terms in the equation could not be characterized. The Oregon Inlet method could not be applied since some of the key input parameters were not available. Generally, the analytic techniques considered require more information than is available for the Atlantic Ocean Channel.

Empirical methods

39. The simplified shoaling rate method and the Volume of Cut approach were not applied, since these techniques require an existing dredged channel so that an extrapolation can be made. Application of the Gole method produced a shoaling rate of 1.7 cm/yr distributed over the proposed Atlantic Ocean Channel or 88,000 m^3/yr. Details of the calculations for the Gole method are given in the addendum to this appendix. Based on the 1980 survey depths (7.5 miles of channel), there is an average of 750,000 m^3/mile of dredging required to initially develop this channel.

40. The method developed by Boicourt (1981), which indicated a threshold velocity of 30 cm/sec for sediment transport, was used as a guide to determine when sediment would move. The technique proposed by Kadib (1976) was not applied, since it includes parameters that could not be quantified for this

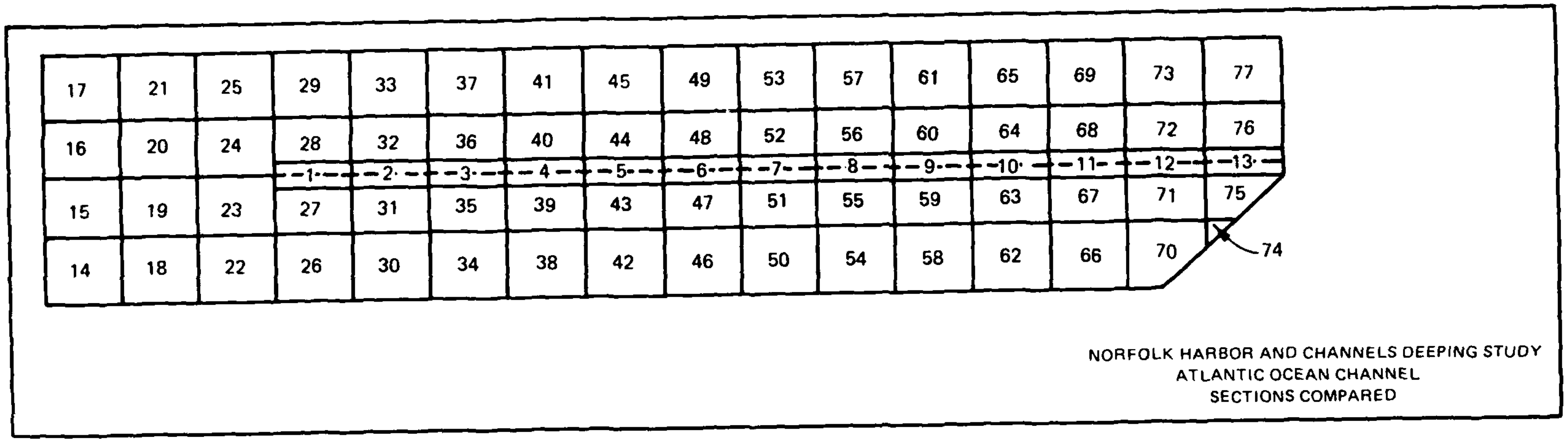


Figure A10. Sections compared

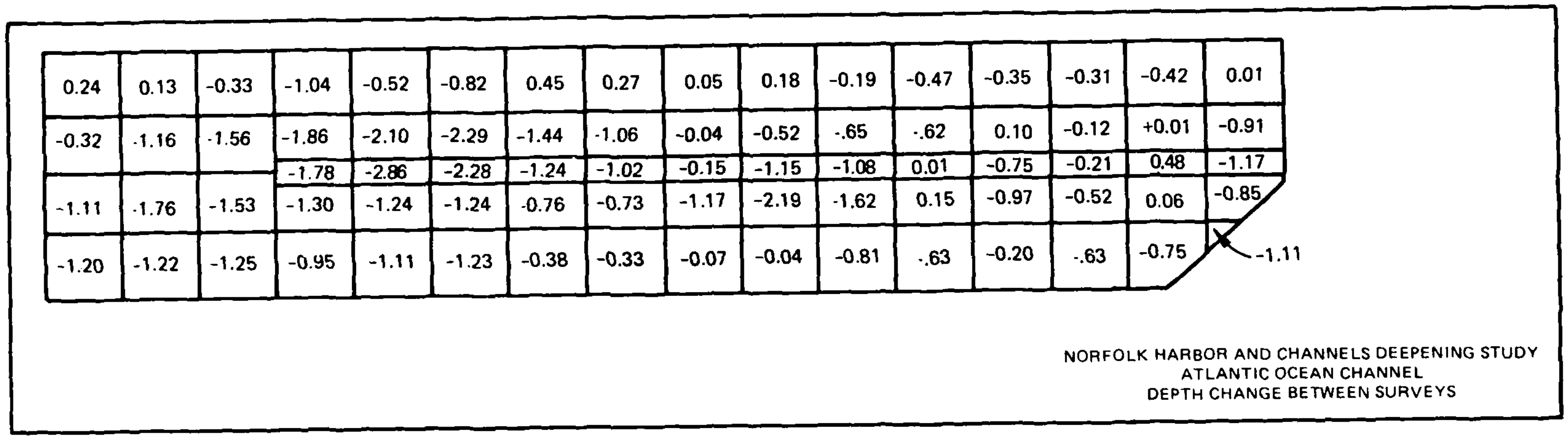
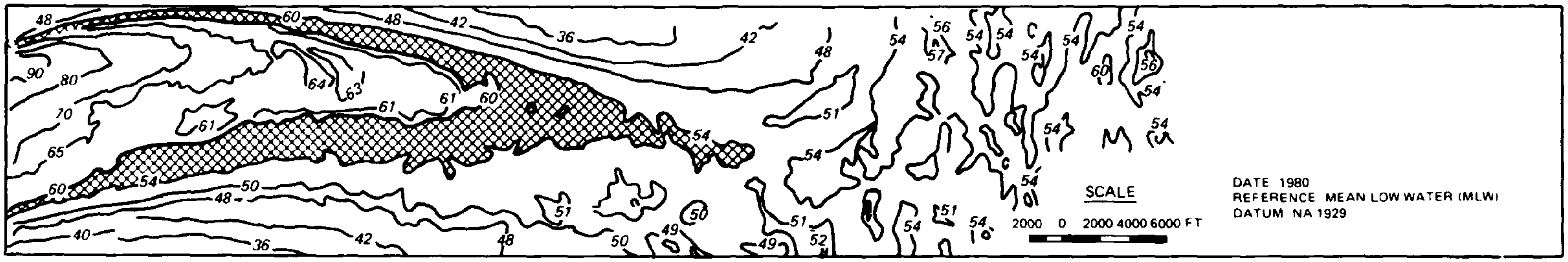


Figure A11. Section data (depth change between surveys, ft)



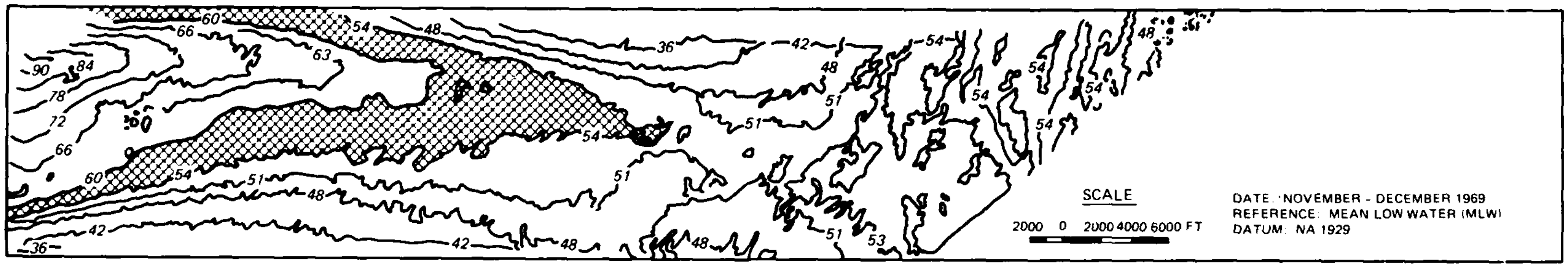


Figure A12. Contour plots

study. Applying the general concepts proposed by Ludwick (1981), a deposition value of 3.1 cm/yr, or 163,000 m^3/yr was determined. Details of the calculations used in the Ludwick approach are given in the addendum.

Summary

41. The analysis of the physical model data indicated that velocities sufficient to move fine sand exist only during spring tides and last for only 2 hr during a tidal cycle. From the analysis of the predicted tidal ranges for 20 years at Virginia Beach, the spring tide sufficient to move sediment exists only approximately 10 percent of the time. Therefore tidal currents acting alone would not cause any significant change in the bed.

42. The analysis of 20 years of wave data indicates that at the present average depth of the channel (56.96 ft) wave orbital velocities are greater than 30 cm/sec 9.3 percent of the time. For the 60-ft Atlantic Ocean Channel, the wave orbital velocities are greater than 30 cm/sec 8.39 percent of the time. Therefore the ability of the waves to place fine sand into suspension exists only during a small portion of the year, and the deepened channel would reduce that duration.

43. One of the most important analyses for this study was the hydrographic survey comparison. In the area of the proposed channel there was a long-term scour. The real significance of this comparison is to demonstrate the relative stability of the area.

Conclusions

44. As a result of the study, it is concluded that:

a. Based on the hydrographic survey analysis, the site of the Atlantic Ocean Channel is stable to slightly deepening at a rate of about 3 cm/yr.

b. The project depth along the proposed channel alignment is at most 10 percent greater than the natural depth. The increased depth will initially cause deposition; and the channel, once dredged, will have an equilibrium depth somewhere between the dredged depth and its present depth.

c. The wave climate is mild. The majority of the waves do not affect bottom sediments. This condition will not be appreciably affected by the deepening.

d. The average rate of infill should be small, ranging somewhere between 2 cm/yr (Gole) to 3 cm/yr (Ludwick). These rates of infill correspond to an annual maintenance dredging requirement in the range of about 115,000 to 200,000 cu yd. For planning purposes, the higher rate is recommended.

Table A1

Chesapeake Bay Entrance 20-Year Wave Summary for All Directions, Station 10

Height	Period, sec										
m	0.0-2.9	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-Longer	Total
0.00-0.49	512	999	836	723	537	645	254	36	78	41	4,661
0.50-0.99		176	654	295	152	663	330	62	163	151	2,646
1.00-1.49			49	215	112	219	126	45	77	81	924
1.50-1.99				15	64	128	59	24	13	27	330
2.00-2.49					14	66	46	15	7	7	155
2.50-2.99						5	17	13	3	6	44
3.00-3.49							1	6	2	4	13
3.50-3.99										3	3
4.00-4.49										1	1
4.50-4.99											0
5.00-greater											0
Total	512	1,175	1,539	1,248	879	1,726	833	201	343	321	8,777

Avg hs(m) = 0.52 Largest hs(m) = 5.17 Total cases = 58,440

Note: Shoreline angle = 28.0 deg azimuth. Water depth = 10.00 m. Percent occurrence (X100) of height and period for all directions. hs = significant wave height.

Table A2

Virginia Beach 20-Year Wave Summary, for All Directions, Station 77

Height	Period, sec										
m	0.0-2.9	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-Longer	Total
0.00-0.49	496	957	813	743	540	629	265	37	57	49	4,586
0.50-0.99		177	634	267	151	681	292	66	193	170	2,631
1.00-1.49			55	232	108	238	115	28	54	53	883
1.50-1.99				19	91	152	63	24	19	15	383
2.00-2.49					18	102	50	15	10	10	205
2.50-2.99						12	43	17	3	4	79
3.00-3.49							3	15	4	2	24
3.50-3.99									3	2	5
4.00-4.49										2	2
4.50-4.99										1	1
5.00-greater											0
Total	496	1,134	1,502	1,261	908	1,814	831	202	343	308	8,799

Avg hs(m) = 0.55   Largest hs(m) = 5.35   Total cases = 58,440

Note: Shoreline angle = 342.0 deg azimuth. Water depth = 10.00 m. Percent occurrence (X100) of height and period for all directions. hs = significant wave height.

Table A3

Section Data Summary

1980, 1969 Average Depths, ft, and Standard Deviation, ft

[illegible] [illegible] [illegible]	2[illegible] 64 5[illegible] [illegible]08 64 6[illegible] 1 [illegible]	25 6[illegible] 89 1 29 6[illegible] 56 1 36	29 60 82 1 84 59 78 1 [illegible]	33 58 89 2.53 58 37 2 11	37 55 78 3.32 54 96 2.70	41 50.90 4.19 51.35 4.23	45 46.75 4.93 47.05 5.08	49 44.09 3.85 44.14 4.15	53 43.77 3.05 43.95 3.12	57 45.51 2.05 45.32 2.12	61 47.65 1.47 47.18 0.95	65 50.84 1 27 50 47 1.32	69 54.12 0 50 53.81 0 51	73 54 42 0 54 54 00 0 00	77 54 04 0 11 54 05 0 31
16 64 29 1 80 61 [illegible] 1 95	[illegible] 6[illegible] 1 29 6[illegible] 1 54	[illegible] 62 [illegible] 1 26 60 80 1 55	28 62 06 0 [illegible] 60 [illegible] 0 80	32 61 32 0 31 59 22 1 21	36 60 17 0.94 57.88 1.48	40 56.66 1.26 55.22 0.48	44 55.41 1.68 54.35 0.51	48 52.62 1.35 52.58 1.24	52 51.44 1.43 50.92 1.19	56 50.57 0.87 49.92 1.06	60 50.83 0.45 50.21 0.90	64 52 18 1 05 52 28 0 83	68 54 04 0.14 53.92 0 19	72 53.98 0 42 53 99 0 03	76 54 95 0 83 54 04 0 31
			1 59 97 0 74 57 69 0 64	2 59 69 0 89 56 83 0 70	3 57.76 1.30 55 48 0.76	4 55.71 0 59 54.47 0.46	5 55.11 0.85 54.09 0.22	6 54.08 0.16 53.93 0.21	7 53.81 0.35 52.66 0.93	8 53.02 0.73 51.94 0.54	9 52.31 0.41 52.32 0.34	10 53.67 0.34 52.92 0 19	11 53.88 0.24 53.67 0.53	12 53.47 0.51 53.95 0.12	13 55 18 0 78 54.01 0 18
15 60 21 1 18 59 10 1 60	19 58 54 1 70 56.78 1 61	23 57 26 1.89 55 73 1.60	27 56.02 1 32 54.72 0.95	31 55 78 1 34 54 54 0.71	35 54.97 0.90 53 73 0.57	39 53.91 0 52 53.15 0.50	43 53.52 0.53 52.79 0.56	47 52.97 0.79 51.80 0.91	51 53.41 0.63 51.22 0.44	55 53 35 0.77 51.73 0.72	59 53.11 0.83 53.26 0.47	63 53.99 0.07 53.02 0.06	67 53 61 0 39 53 09 0 70	71 53 34 0.43 53 40 0.63	75 54 49 1.20 53 64 0 44
14 53 41 2 77 52 21 2 58	18 51.71 2 40 50.49 2.26	22 50.93 2.11 49.68 2 06	26 51 11 1 90 50 16 1 81	30 51.67 1.50 50.56 1.67	34 51.86 1.22 50.63 1.20	38 51.49 0.80 51.11 0.76	42 51 19 0.51 50.86 0.81	46 50.27 0.37 50.20 0.92	50 50.70 0.69 50.66 0.58	54 50.58 0.83 49.77 1.18	58 52 28 1.15 51.65 1.23	62 53.05 1.04 52 85 0.44	66 53.23 0.93 52.60 1.45	70 51 79 1.01 51.04 1.29	74 53 39 0 43 52.28 0.21

Table A4

Section Data Summary

Volume Change, yd^3, and Depth Change Between Surveys, ft

17 66,889 +0.24	21 36,815 +0.13	25 87,889 -0.33	29 286,444 -1.04	33 136,815 -0.52	37 219,148 -0.82	41 121,889 +0.45	45 73,519 +0.27	49 13,556 +0.05	53 49,185 +0.18	57 53,296 -0.19	61 126,185 -0.47	65 97,852 -0.35	69 86,111 -0.31	73 115,519 -0.42	77 2,778 +0.01
16 81,148 -0.32	20 309,852 -1.16	24 409,630 -1.56	28 406,630 -1.86	32 461,407 -2.10	36 509,481 -2.29	40 322,481 -1.44	44 230,000 -1.06	48 7,741 -0.04	52 111,222 -0.52	56 143,778 -0.65	60 132,815 -0.62	64 20,889 +0.10	68 25,630 -0.12	72 3,630 +0.01	76 189,963 -0.91
			1 200,741 -1.78	2 322,667 -2.86	3 287,852 -2.28	4 139,444 -1.24	5 118,481 -1.02	6 17,519 -0.15	7 131,222 -1.15	8 126,222 -1.08	9 1,333 +0.01	10 86,111 -0.75	11 25,667 -0.21	12 58,074 +0.48	13 142,926 -1.17
15 289,074 -1.11	19 467,111 -1.76	23 401,704 -1.53	27 265,296 -1.30	31 250,815 -1.24	35 250,667 -1.24	39 155,519 -0.76	43 151,741 -0.73	47 247,000 -1.17	51 451,862 -2.19	55 343,111 -1.62	59 30,444 +0.15	63 203,704 -0.97	67 106,074 -0.52	71 13,593 +0.06	75 137,333 -0.85
14 353,481 -1.20	18 367,889 -1.22	22 363,444 -1.25	26 276,370 -0.95	30 316,556 -1.11	34 350,333 -1.23	38 109,778 -0.38	42 93,370 -0.33	46 20,852 -0.07	50 10,667 -0.04	54 227,444 -0.81	58 176,889 -0.63	62 57,407 -0.20	66 176,185 -0.63	70 193,000 -0.75	74 30,148 -1.11

ADDENDUM: ATLANTIC OCEAN CHANNEL CALCULATIONS

Gole Method

1. The Gole method is an empirical method developed for the prediction of siltation in harbor basins and approach channels. The method is designed to treat suspended loads and is based on the analysis of prototype and model data and theoretical studies.

2. Included in the assumptions made for developing this method are the following:

   a. The suspended sediment capacity of current is proportional to the square of velocity.

   b. The mechanism of turbulence that really keeps silt in suspension is not considered.

   c. Flow is assumed to be perpendicular to channel alignment. (Based on the physical model observations, the flow in the vicinity of the proposed channel tends to be parallel to the channel. However, there are sufficient periods of time during the tidal cycle in which flow across the channel exists to justify the application of this technique.)

3. The equation is as follows:

$$S = K\ L\ v\ C\ t\ d\left(\frac{BV_o}{Vd}\right)\left(\frac{D^2 - d^2}{D^2}\right)$$

where

S = effective silt load $\left[ML/T^2\right]$

K = empirical coefficient (use 0.31 for approach channels)

L = channel length $[L]$

v = water velocity approaching channel $[L/T]$

C = silt concentration in water column $\left[M/L^3\right]$ approaching channel

t = time $[T]$

d = depth of flow approaching channel $[L]$

B = width of the channel

V_o = particle fall velocity $[L/T]$

V = water velocity across channel

D = channel depth $[L]$

4. The specific values used to make the estimate for siltation were:

$L = 17{,}059$ m

$v = 0.46$ m/sec (average representative value)

$C = 0.007$ kg/m^3

$t = 3.16 \times 10^7$ sec (1 year)

$d = 15.90$ m (52.15 ft)

$V = 0.40$ m/sec

$D = 18.29$ m (60 ft)

$B = 304.8$ m (1,000 ft)

$V_o = 0.001$ m/sec

Using these values

$$S = 100 \times 10^6 \text{ kg annually}$$

Assuming that the density of the deposited material is 1134 kg/m^3, the volume of siltation is 88,000 m^3 annually, or an average over the channel of 1.7 cm/yr.

Ludwick Method

5. The Ludwick method is based on the expression that approximates the deposition of fine-grained sediment by settling in the presence of a moving current

$$\frac{dm}{dt} = \bar{c}\,\bar{w}\left(1 - \frac{\bar{L}o}{\bar{L}c}\right) \quad \text{For deposition} \left(\frac{\bar{L}_o}{\bar{L}_c}\right) < 1$$

where

m = mass of sediment deposited per unit area of bed $[m/L^2]$

t = time $[T]$

$\bar{c}$ = depth-mean concentration of suspended sediment $[L/T]$

$\bar{w}$ = mean settling velocity of suspended sediment $[L/T]$

$\bar{L}o$ = shear stress of bed, due to moving sediment and water $[F/L^2]$

$\bar{L}c$ = critical shear stress above which sediment cannot be deposited $[F/L^2]$

6. If it is assumed that when $\bar{L}o < \bar{L}c$, $\bar{L}o/\bar{L}c = 0$, then integrating yields $m = \bar{c}\,\bar{w}\,t$. If this mass of deposited sediment is transformed into a

thickness accumulation rate R , where $R = m/\rho st$,

then

$T = (\bar{c}\ \bar{w} / \rho_s)$ (Ludwick 1981)

$\bar{c}$ = depth-mean concentration of suspended sediment in the water $[M/L^3]$

$\bar{w}$ = mean settling velocity of suspended sediment [L/T]

ρ_s = in-place bulk density of sediment $[M/L^3]$

Values of constants (Ludwick 1981):

$\bar{c} = 7 \times 10^{-6}\ g/cm^3$

$\bar{w} = 0.1\ cm/sec$

$\rho_s = 1.134\ g/cm^3$

$R = 7 \times 10^{-6}\ g/cm^3 \times 0.1\ cm/sec/1.134\ g/cm^3$

R = 6.17 E-07 cm/sec

R = 2.22 E-03 cm/hr

Assume deposition occurs only during the time of slack water for only 1 hr.

$$\frac{365\text{-}1/4 \text{ days} \times 24 \text{ hr/day}}{6.21 \text{ hr}} = 1{,}411.59 \text{ slack waters per year}$$

2.22 E-03 cm/hr × 1,411.59 hr = 3.1 cm/yr

This results in a prediction of 163,000 m^3/yr.

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